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FEASIBILITY OF A STABILIZATION-RETENTION BASIN IN LAKE ERIE AT CLEVELAND, OHIO

MAY, 1968

Prepared For

FEDERAL WATER POLLUTION CONTROL ADMINISTRATION

Under

CONTRACT NO. 14-12-27

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HAVENS AND EMERSON
Consulting Engineers

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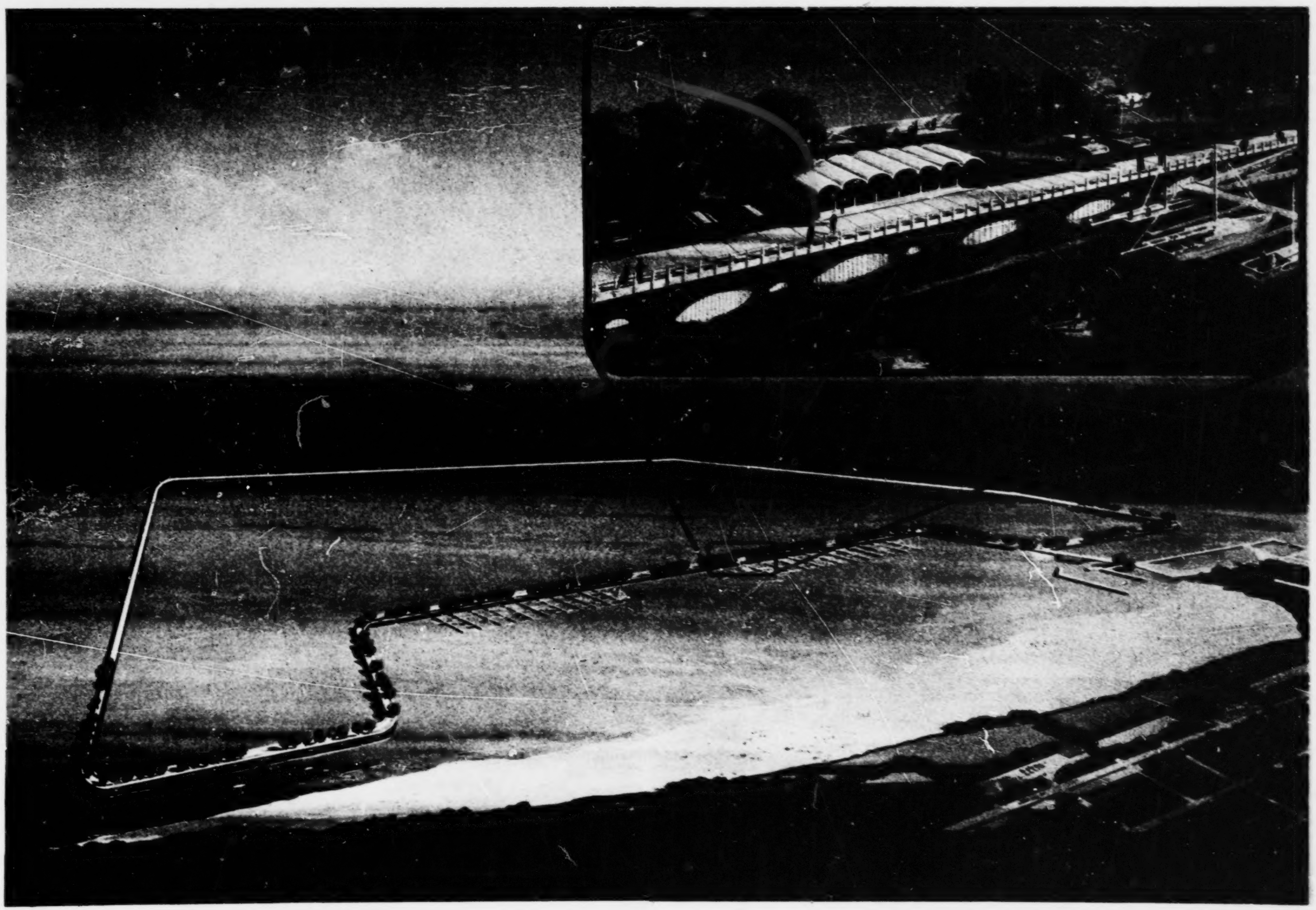
Cleveland   New York



OFFSHORE STABILIZATION BASIN HAVENS & EMERSON · CONSULTING ENGINEERS

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ABSTRACT

A feasibility study was conducted of a large stabilization-retention basin to be constructed in Lake Erie at Cleveland, Ohio. The stabilization is viewed as a possible alternative to separation of a combined sewer system. The proposed basin would treat flows from a number of large combined sewer overflows, from several polluted streams, and effluent from a large secondary wastewater treatment plant. Treatment would consist of bio-oxidation, sedimentation, stabilization and disinfection. A shoreline collection system is included to convey flows to the basin.

The chemical, biological, physical and structural aspects of the proposed basin were studied, and the probable benefits to water quality and the effectiveness of the basin as a treatment device were evaluated. Estimates of cost of the basin and collection system were prepared, and it was concluded that the stabilization basin would provide a higher degree of pollution abatement than would separation of sanitary and storm sewers, at about one-third the cost.

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

A. SUMMARY

1. The physical characteristics, population, pollution loadings and hydrological characteristics of the study area have been determined, and are presented in Part 2 of the report.
2. Runoff hydrographs of the streams and drains serving the study area have been obtained for the design storm, and flow routings have been computed to determine rates and volumes of flow to be used for design at various points of the collector system and for the stabilization basin. These are tabulated in detail in Part 2 of the text.
3. The input loads of polluting materials from all sources have been established and are tabulated in Table 2-6. Among these loads are total monthly inputs as follows:

BOD_5	1,400,000 lbs/30 days
COD	4,660,000 lbs/30 days
SS	2,120,000 lbs/30 days
Flow Volume	24,800 acre-ft/30 days

4. The general design objectives have been defined, and past experience in existing impoundments having related features has been surveyed as a guide to objectives, performance and potential problems of the proposed project.
5. Basic design criteria for the stabilization basin have been established as follows:
   (a) Overall detention period to be 30 days, based on average flows from treatment plant, combined sewer

overflows and surface streams, plus runoff and overflow from a 3-day storm of intensity expected to occur once in three years.

(b) Volume of basin to be 30,000 acre-feet, which is the total volume required by (a) plus allowance for displacement by wave action, increase in future runoff rates and hydraulic inefficiencies.

(c) Surface area of basin to be 900 acres, and mean depth to be approximately 34 feet.

(d) Basin to be subdivided into three zones: an aerobic, mixed zone for bio-oxidation; a facultative quiescent zone for sedimentation; and a reaeration and chlorine contact zone prior to discharge of effluent.

(e) Structural design of basin wall to comprise a cellular steel sheet piling cofferdam, filled with sand, crushed stone or other suitable fill. Cell diameter and fill characteristics are to be determined after completion of detailed design studies based on foundation and sub-soil investigations. Preliminary design based on 67-foot-diameter cells, having 10-foot freeboard, concrete capped. Approximately 410 cells are required.

6. Basic design criteria for the shoreline collector system have been established as follows:

(a) Hydraulic design flow based on peak runoff rate occurring from a 1-year storm, after routing to the point of design.

(b) A combination of gravity flow conduits with pumping stations and force mains was found to be the most economical collection system, and is recommended in preference to an all-gravity system. Four pumping stations are required, (including the Easterly effluent lift station).

(c) The most economical route for the collector conduits and force mains is at the beach line rather than in tunnel or in deep cut in the shoreline bluff.

(d) At collector basins, trash removal and chlorination of excess flows will be provided.

B. CONCLUSIONS

7. The operation and maintenance features of the system have

been studied in preliminary form as follows:

(a) The collector system will require only routine operation and maintenance of pumps, trash racks and chlorination facilities.

(b) The first-stage aerobic zone should be artificially aerated and mixed to maintain satisfactory conditions for biological oxidation. Preliminary design indicates that three 40-horsepower mechanical or diffused air pumps will be required.

(c) The stabilization basin will require regular surveillance and maintenance for control of algae growth, removal of floating debris and scum, and perhaps occasional use of algicides to prevent nuisance blooms.

(d) The net sludge accumulation should either be removed from the basin or sealed from the overlying water to prevent re-solution and release of nutrients. Sludge removal once in 5 to 10 years is indicated. Removal of sludge to the proposed dredged spoil impoundment is the preferred method of sludge disposal.

8. The anticipated benefits of the project in terms of pollution abatement and secondary effects are:

(a) Reduction of polluting materials referred to input concentrations is estimated as follows:

Constituent	Reduction, %
BOD_5	70
TS	60
SS	72
COD	64
Total phosphate (as P)	50
Total nitrogen (as N)	50

These estimated reductions would increase the overall removal of BOD in the Easterly Plant from about 89 percent to 96 percent, and suspended solids from 85 percent to 97 percent.

(b) The basin would provide valuable protection as a breakwall against erosion of over 1.5 miles of the Cleveland and Bratenahl shoreline. Cost of equivalent shoreline protection as estimated by the Corps of Engineers is about $12,500,000 at 1968 price levels.

(c) The project would create a sheltered bay about 1.2 miles long which would be an invaluable recreational asset, suitable for swimming, boating, fishing and other water-based recreation. This bay would be protected physically, and water quality could be readily controlled to bathing water standards.

(d) The stabilization basin would complement the proposed Corps of Engineers impoundment for dredging spoil. The two projects could be mutually beneficial, and construction with a common wall would result in an overall cost reduction of about $10,000,000.

(e) The basin would replace a chlorination contact chamber required for the Easterly Plant, presently estimated to cost $3,000,000.

(f) Other benefits are possible, such as providing a site for further nutrient removal, possibilities as a research tool for aquatic biology, algae control, wildlife management and others.

9. Preliminary estimates of costs for the full project including the stabilization basin and both East and West Collector systems are:

Total Construction Cost	$72,600,000
Total Project Cost*	83,500,000
Annual O & M Cost	230,000
Total Annual Cost, incl. Amort. plus O & M	4,767,000

10. Preliminary estimates of project costs for the recommended first-stage project including the stabilization basin and West Branch Collector, exclusive of the East Branch Collector are:

Total Construction Cost	$64,300,000
Total Project Cost*	74,000,000
Annual O & M Cost	167,000
Total Annual Cost, incl. Amort. plus O & M	4,188,000

*Rounded Construction cost plus 15% for contingencies, easements, engineering, testing, supervision of construction, legal and administrative costs.

11. On the basis of this preliminary study, the proposed project has been found to be feasible from the standpoint of fulfilling its primary and secondary objectives. Conclusions regarding feasibility are briefly summarized as follows:

(a) The basin is feasible as a treatment device in reducing pollution of Lake Erie.

(b) The project will accomplish a higher degree of pollution abatement than sewer separation, at about one-third the cost.

(c) The basin is feasible from the standpoint of structural design and construction.

(d) The project can be operated and maintained at reasonable cost, without extraordinary or especially difficult operating problems.

(e) The basin will not create a significant navigation problem.

(f) The project will have a beneficial effect on adjacent properties along the shoreline, in protection from beach erosion and in recreational benefits.

(g) No major objections or obstacles which would prevent its construction have been found.

12. Viewed as an alternative to sewer separation, the proposed stabilization-retention basin is economically feasible. Other studies have shown that the cost of separation of

sanitary and combined sewage flows in Cleveland would be about $21,400 per acre, totalling $278,000,000 for the study area under consideration. The total construction cost of the stabilization basin project is estimated at about $83,500,000, or about 30 percent of the cost of separation.

13. The proposed basin project would accomplish a greater degree of pollution abatement than sewer separation in Cleveland, by collecting and treating all of the surface drainage originating in the service area. Data indicates that about one-third of the dry weather pollution load carried by the streams originates in the suburbs. This load would not be affected by the separation of Cleveland's combined sewers, but would be substantially reduced by the basin project.

C. RECOMMENDATIONS

14. A smaller scale basin designed to treat only the Easterly Plant effluent and overflow was determined to be infeasible from the standpoint of cost and overall effectiveness in pollution abatement, and is not recommended.

15. It is recommended that the proposed stabilization basin project with the associated shoreline collector systems be adopted as the plan for pollution abatement for the metropolitan Cleveland system east of East 79th Street. This project should be included in Cleveland's overall Master Plan for Pollution Abatement, and should be undertaken without delay.

16. It is recommended that the project be undertaken in two stages. Stage one would include the stabilization basin and the West Branch Collector System. Stage two would add the East Branch Collector System at a future date when required by increase in pollution loads.

17. It is recommended that detailed engineering design work be undertaken immediately, including:

    (a) Field surveys, subsoil and foundation studies.

    (b) Model studies for determination of wave and ice forces to be used for detailed design.

    (c) Detailed design and preparation of construction plans and specifications, and detailed estimates of cost.

PART 1

INTRODUCTION

Separation of sanitary sewers and storm drains has been widely recommended as a means of reducing pollution of Lake Erie and other receiving waters throughout the country where present sewerage systems are on the combined plan. This method of pollution abatement has some merit but suffers from two serious defects. First, the extremely high cost makes this method prohibitive from the standpoint of traditional methods of municipal financing, and no suitable alternative financing plan has been put forward. Second, the separation of sewage from storm runoff is not fully effective as a means of abating pollution; studies have shown that significant amounts of pollution in the form of coliform organisms, BOD, nutrients, and other polluting materials are contained in storm water runoff from separate systems discharged to receiving waters.

Therefore, it is highly desirable to find alternatives to the complete separation of sewers which would be less costly and be equally or more effective in reducing pollution from combined sewer overflows. This report deals with the feasibility of one such possible alternative.

The inshore waters of Lake Erie at Cleveland are polluted by three types of discharges: overflows from combined sewer interception devices which discharge at the beach line; flow from storm drains and polluted surface streams; and effluent from two wastewater treatment plants located on the lake shore. It is these three sources of pollution which are to be attacked by the project which is the subject of this study.

We consider herein the feasibility of constructing a large impoundment in the offshore waters of Lake Erie to serve as a retention and stabilization basin. The proposed basin would serve a specific area on the east side of the metropolitan Cleveland sewerage district; other programs are being developed to abate pollution in the harbor area and the westerly district.

The influent flow to this basin would consist of effluents and overflows from the Easterly Wastewater Treatment Plant, plus all combined sewer overflows now discharging directly to the Lake, plus flows from all major storm drains and surface streams within the study area. Flows from these sources would be concentrated at collection points at the lake shore by means of intercepting channels or conduits constructed roughly parallel with the shore. At the collecting points, facilities would be provided for screening of coarse suspended materials such as tree limbs and similar debris, and for skimming and removal of floating scum and oil. Flow from the collection points would be discharged continuously to the stabilization basin. Easterly Treatment Plant effluent would be discharged directly into the basin for treatment prior to discharge into Lake Erie.

In essence, the effect of this plan is to collect and treat the entire contribution of wastewaters and surface runoff from a highly urbanized area before these flows are released to the receiving waters.

The report which follows is a study of the feasibility of this plan, and considers the sanitary engineering aspects, hydraulic and structural features, the biological effects, and aesthetic considerations, and various secondary effects of the proposed stabilization basin.

PART 2

COLLECTION OF DESIGN DATA

The collection of design data was a substantial part of this project. In order to evaluate the character of the study area for the design of the offshore stabilization basin, it was necessary to collect background data on the quantity and quality of water coming from the watershed. Other basic data, such as area and population benefitted, was also required to evaluate the feasibility.

1. Description of Study Area

The study area adopted for this project comprises 38,800 acres of urban and suburban area on the east side of the Metropolitan Cleveland Service District. Thirty percent of the study area is within the City of Cleveland, and about thirty-three percent of the study area is served by combined sewers. The study area and its relationship to the Cleveland sewerage district is shown on Fig. 2-1. The dry weather sewage flow from the study area is collected by the Easterly Interceptor and the Lake Shore Boulevard Interceptor for delivery to the Easterly Treatment Plant. Both of these two large interceptors are parallel to and close to Lake Erie. The storm water and combined sewer overflow from the entire study area reaches the Lake, either through a storm drain or one of the natural drainage courses. There are 11 major sewer subdistricts within the study area, for which the pertinent characteristics are listed in Table 2-1. It is the drainage

TABLE 2-1

DRAINAGE AREA

Drainage Area	Total Area Acres	Area at Gauge Acres	Length of Channel ft.	Average Slope ft./mi.	Acres of Open Space	Percent Impervious	Population
East 79th	1,510	1,490	23,000	-	0	58	50,300
East 88th	406	404	8,800	-	0	60	17,100
Doan	7,230	6,950	47,000	60	1,820	42	99,200
Dugway	5,630	5,500	36,000	60	1,040	42	110,600
Shaw	680	680	14,000	66	0	63	14,300
Nine Mile	5,080	5,030	37,000	66	700	42	64,800
East 156th	380	380	9,900	-	80	49	5,900
Green	660	660	20,000	80	125	31	5,100
Neff	87	87	4,200	-	0	50	1,400
Euclid Creek	15,500	15,000	43,000	64	8,690	15	63,800
East 140th	1,640	1,640	14,400	-	0	56	21,000
	38,800						453,500

from these areas, as well as the Easterly plant effluent that causes most of the pollution of the beaches and recreational areas of Lake Erie in the vicinity of Cleveland.

The high bacterial counts of the inshore waters render the recreational water unfit for body contact water sports.

The portion of the study area served by combined sewers is the older section of the Cleveland area. The system has been expanded and extended beyond the expected design of fifty years ago. Sewers from the suburbs empty into the upstream end of these combined sewers increasing the dry weather flow throughout the entire length. Increased populations beyond the per capita design factor, and the great increase in water use in the past fifty years have increased dry weather flow to the point where many overflows function during periods of maximum daily flow in dry weather. The increase in paved areas, connection of downspouts, lack of maintenance personnel and construction of additional inlet basins have all increased the storm flow in the sewer and consequently have increased the frequency of overflow. Most of the suburban systems, being of more recent design and construction have been constructed as separate systems. However, illicit connections between storm and sanitary sewers and cross connections for relief of sanitary sewers have led to a reduction of storm water quality from the separate storm sewers. It has also been shown that separate storm runoff carries a substantial pollution load. Studies by Weibel, et al(1)(2) have shown solids, BOD, and bacterial concentrations of storm runoff to be in the same general range as combined sewer overflow. Results of work by Havens and Emerson in Cleveland on gutter samples have also shown high solid contents, as given in Table 2-2.

TABLE 2-2

GUTTER SAMPLES

(Figures in mg/l)

Parameters	Number of samples	Range	Mean	Median
Total Solids	5	800 - 1941	1180	1104
Suspended Solids	6	364 - 904	705	833
BOD	7	29 - 171	79	75
NO_3 (as N)	8	2.12 - 29.2	11.4	6.4
PO_4 (as P)	7	1.74 - 39.8	11.9	2.9
pH	9	5.1 - 8.0	-	7.0
SO_4	4	25 - 470	226	205

The study area contains a population of 453,500. Some of the highest densities of population in the City of Cleveland occur in the study area. The far east side of the study area on the other hand, has some of the lowest population densities. The population of each of the subdistricts was computed and is given in Table 2-1 with the subdistrict figures.

Contour maps of the shoreline were not available for use in the design of the collection system and had to be prepared from aerial photographs. Topographic maps were prepared with a scale of 1 inch equal to 200 feet and with 5 foot contour intervals. Also for the collection system design, several test borings were made along the tentative sites to allow more accurate consideration of location and cost.

2. Hydrology

The collection of rainfall and runoff data provided basic data for the basin and collection system design. The volume of the basin and size of the collection system depends upon the runoff resulting from rainfalls. In this section of the report, rainfall and the concurrent runoff will be discussed.

(a) Rainfall. The total depth and intensity of rainfall is important in design of basin volume and capacity of collection system. The total depth of rainfall and the concurrent runoff establishes the volume of the basin allocated to retain the surface water runoff. The rainfall data is also needed to arrive at percentages of runoff from the various areas, and the intensity of rainfall and its effect upon peak flows of runoff is required for the design of the size of the collection system.

Available data from nine recording rain gauges over the City, six of which were located within the study area, provided the necessary information of rainfall quantity and intensity for constructing unigraphs and for study of the runoff characteristics. Long term data from the official Cleveland Weather Bureau gauge at the Cleveland Hopkins Airport provided the data for frequency - depth - duration studies. Using the same station data, the design storms were developed. The shape and duration of the design storm was investigated from the Cleveland Hopkins Airport gauge. The variation of rainfall over a large area was examined using all of the rainfall gauges. The results of this work led to the evaluation of runoff characteristics and the design storm rainfall mass diagrams by which the collection system was designed.

Details of the work and figures are given in Appendix A, Hydrology.

(b) Runoff. Measurement of the runoff, both dry weather flow and storm flow, was necessary to provide the volume requirements of the basin as well as to determine the runoff characteristics of each watershed for the design of the collection system. All of the major discharge points of pollution to Lake Erie were gauged, a rating curve developed by field observation and velocity tests, and the results compared to rainfall for runoff character analyses. The results of this runoff gauging program are shown in Table 2-3 and Table 2-4. Additional details of the gauging and description of the gauging sites may be found in Appendix A. The stream and combined sewer overflow gauging program provided continuous dry weather flow information and hydrographs resulting from rainfall. These hydrographs were used to construct a unitgraph for the streams and to obtain runoff characteristics of the drainage areas which could be used with the design storm rainfall data to develop the design storm hydrographs.

TABLE 2-3

PERTINENT HYDRAULIC DATA

Stream	Drainage Area @ Mouth Acres	Drainage Area @ Gauge Acres	Date Gauge Installed	Date Gauge Removed	Period of Gauging, Weeks	Depth of Rainfall Before Runoff Commences	Basin lag (Min.)
Doan	7,231	6,950	8/ 2/66	10/ 4/67	61	.02"	60
Dugway	5,630	5,500	8/23/66	8/31/67	53	.02"	45
Shaw	680	680	3/10/67	9/11/67	26	.02"	45
Nine Mile	5,078	5,000	3/29/67	8/16/67	20	.02"	90
Green Creek	657	657	3/ 9/67	9/18/67	27	.02"	55*
Euclid	15,500	15,000	2/28/67	9/22/67	29	.02"	135
Combined Sewer Overflow						Before Overflow Begins	Time of Concentration (Min.)
E. 79th	1,510	1,510	3/15/67	9/ 7/67	25	.05"	63
E. 88th	406	404	3/15/67	9/ 7/67	25	.03"	49
Easterly	1,641	1,544	3/15/67	9/10/67	25	.02"	55
E. 156th	383	380	3/ 8/67	9/18/67	27	.01"	46
Neff Road	87	71	3/18/67	9/ 7/67	24	.02"	37

*Time of Concentration.

TABLE 2-4

MEAN DAILY DISCHARGES OF STREAMS AND DATA COLLECTED FROM GAUGING

Stream	Discharge Mean Daily cfs	Discharge Mean Daily cfs/acre	Maximum Daily cfs	Maximum Peak cfs
Doan	12	.0018	105	800
Dugway	8	.0015	110	420
Nine Mile	17	.0034	68	1,000
Shaw	2.2	.0032	7	170
Euclid	25	.0017	180	1,300

3. Hydrological Results

Using the collected rainfall and runoff data, the characteristics of each sub-district watershed could be evaluated. Dry weather flow and the runoff characteristics of storms were evaluated. Unit hydrographs were developed for the streams, and hydrographs for the combined sewers were developed by a modified Chicago Hydrograph Method referred to as the routed hydrograph method. Details of this work are presented in Appendix A.

(a) Streams. Unitgraphs were developed for Doan, Dugway, Shaw, Nine Mile, and Euclid Creeks. Several good hydrographs resulting from storms, along with the continuous rainfall data available allowed unitgraphs to be developed by the successive approximation technique. The derived unitgraph was used with the design storm rainfall to produce design storm hydrographs for peak discharge rates and volumes of runoff.

(b) Combined Sewer Overflow. Using the modified Chicago Hydrograph Method and runoff characteristics of the sub-district drainage areas, the design storm hydrographs were computed to obtain peak discharge rates and volume of runoff.

Table 2-5 shows the resulting peak discharges from the input points. The peak discharge rates for all input points were computed by the rational method and the routed hydrograph method for comparison, and these data are also shown in Table 2-5.

4. Water Quality Data

The water which will be conveyed to the offshore stabilization basin comes from three sources: combined sewer overflow; polluted surface streams; and effluent from the Easterly Wastewater Treatment Plant, an activated sludge plant. The sludge from this plant is pumped to another site for disposal, so there is no digester supernatant liquor involved at this plant site. The pollution input from these three sources was evaluated.

(a) Quality of Water from Runoff. All of the input loads to the proposed basin were sampled for water quality. Both low flow and storm runoff quality were sampled by a program established on a regular schedule twice weekly and again at any times of increased flow, due to rainfall. The streams, of course, had a continuous flow and were sampled on a routine basis, but the combined sewer overflow could only be sampled during storm runoff. Sampling days were varied to include all days of the week. The routine samples were collected in the morning and

TABLE 2-5

RUNOFF DISCHARGE OF STREAMS AND COMBINED SEWER OVERFLOWS RESULTING FROM 1-YEAR STORM RAINFALL

	Rational Method Discharge		Routed Hydrograph Discharge		Unit Hydrograph Discharge	
	cfs	cfs/acre	cfs	cfs/acre	cfs	cfs/acre
Doan Brook	1,185	.171	1,283	.185	1,330	.192
Dugway Brook	2,460	.477	2,260	.411	1,385	.252
Shaw Brook	523	.769	506	.744	557	.814
Nine Mile Creek	1,762	.353	1,745	.349	1,130	.226
Euclid Creek	4,260	.275	3,330	.215	1,867	.12
Green Creek	357	.540	335	.507		
E. 79th St.	866	.574	815	.540		
E. 88th St.	302	.748	288	.712		
E. 140th St.	957	.619	910	.589		
E. 156th St.	274	.721	270	.710		
Neff Road	61	.872	61	.872		

delivered to the laboratory within 4 hours. Separate bacteriological samples were taken with sterile techniques, and samples for chemical analysis were not fixed since the time between sampling and analysis was short. The quality parameters analyzed were five-day BOD, COD, Kjeldahl nitrogen, total phosphates, suspended solids, total coliform, fecal coliform, and fecal streptococci. BOD, suspended solids, total phosphates, COD, and fecal coliforms were run on all routine samples. Total coliform and fecal streptococci were run on every other routine stream sample. During storm runoff all of the parameters were run on all samples. Sampling was started on February 8, 1967 and continued until July 26, 1967. During this time, a total of 52 sample runs were made.

All analyses were done in accordance with Standard Methods (Twelfth edition) by a commercial laboratory. Duplicate samples submitted indicated standard deviation was within limits of analytical accuracy. Bacteriological work was done with at least four dilutions and also was in accordance with Standard Methods (Twelfth edition) Membrane Filter Techniques, except for fecal coliform which was done by the method described in the FWPCA Publication WP-20-3, "Sanitary Significance of Fecal Coliforms in the Environment."

Results of the chemical and bacteriological analyses were submitted immediately after completion of the five-day BOD test. The results were tabulated on summary sheets and plotted on graphs showing concentration versus time to indicate variation with past sampling. At the end of the sampling period, all of the chemical data was summarized by a computer program which evaluated the data to give the number of samples, the arithmetic mean, median value and range. The results are shown on Table A-7 and A-8, pages 128 and 129. Median values are given in Table 2-6.

Havens and Emerson had sampled these same streams during the summer of 1966 in connection with other studies. The sampling was made bi-weekly for the three month period of the summer, for a total of about ten samples. This data was available and was compared to the data collected for this study. The alkalinity and nitrate data from this source is included in the load data. Changes to the sewer system brought about partially by findings of the 1966 sampling and by earlier studies changed the quality of some of these streams substantially. For example, repair of overflows in the Doan Brook system eliminated about 5 mgd of raw sewage which had previously been overflowing into the creek. Other changes, not as considerable, made slight changes between 1966 and 1967.

(b) Easterly effluent. Quality data of the Easterly Wastewater Treatment Plant effluent were available from the routine plant laboratory analyses. The laboratory at the plant also used Standard Methods for all of the analyses included in this particular project. In order to minimize the range and to get the representative long term data, the average monthly values were used for years 1966 and 1967 to compute basin loads.

During the study it became evident that the Easterly effluent would be the largest single load of pollutional material to the basin. In order to arrive at a better estimate of the

TABLE

BASIN

	AVG. Q cfs	MEAN 30-DAY VOLUME ACRE-FT.	COD mg/l	BOD_5 mg/l	SS mg/l	PHOSPH. (P) mg/l	KJEL NITROGEN (N) mg/l	NITRATE as N *1 (N) mg/l
1. BASE FLOW RUNOFF:								
DOAN	12	714	65	9.0	52	0.71	2.5	0.8
DUGWAY	8	476	92	21.3	30	1.50	7.2	0.9
SHAW	3	179	56	7.0	32	0.97	1.7	0.9
NINE-MILE	17	1011	64	11.1	27	1.07	3.9	0.8
EUCLID	25	1488	51	5.4	73	0.36	1.5	1.0
SUB-TOTAL		3,868						
2. COMBINED SEWER OVERFLOW:								*2
E. 79		9	384	61.5	211	2.24	8.8	0.1
E. 88		17	260	52.4	114	2.16	10.6	0.1
EASTERLY OVERFLOW		275	310	53.1	126	2.53	13.1	0.1
E. 156		14	140	31.2	85	1.6	6.2	0.1
GREEN CR.		12	93	11.7	130	0.43	2.3	0.2
NEFF RD.		2	64	7.4	33	0.16	0.8	0.1
SUB-TOTAL		329						
3. EASTERLY S.T.P. EFFL. (2000)							NH_4	
YR. 2000 -(151 mgd)							as N *3	
MEAN FLOW, EFFL.	234	13,900	75	21.2	24	4.2	6.9	0.8(NO_3) 0.1(NO_2)
4. STORM FLOWS:								
(RUNOFF EXCESS FROM PEAK - (Separate)		3312	35	15	30	0.7	3.0	0.1
(Consec. 3 - DAYS, 3 YR. FREQ.. (Combined)		1250	70	30	45	1.0	6.0	0.1
TOTAL		4562						
EXCESS S.T.P. FLOW		2,100	100	50	50	1.0	6.0	0.1
SUB-TOTAL		24,759						
TOTAL, (1-2-3-4)								

*1 From Data of City of Cleveland, USGS, FWPCA.

*2 Actual values less than 0.1 (except Green Cr.)

*3 Total KJEL Nitrogen - NH_3 - Organic N.

LE 2-6

N LOADS

TE 1	TOTAL N mg/l	ALK. as Ca CO$_3$ mg/l		LOADING - TOTAL POUNDS - 30 - DAYS							
				COD	BOD	SS	P	KJEL N	No$_3$ N	TOTAL N	ALK.
	3.3	108		127,000	17,600	101,900	1390	4900	1560	6400	210,000
	8.1	141		119,200	27,600	38,800	1950	9330	1200	10,500	183,000
	2.6	100		19,500	2,400	11,200	340	550	310	910	34,800
	4.7	90		176,000	30,500	74,300	2900	10,700	2200	12,900	247,000
	2.5	84		207,000	21,900	296,000	1470	6,090	4050	10,100	340,000
				648,700	100,000	522,200	8,050	31,570	9,320	40,810	1,014,800
2											
	8.9			9,600	1540	5,280	56	216	2	218	
	10.7			12,000	2400	5,260	100	490	5	495	
	13.2			232,000	40,000	94,600	1900	9800	75	9875	
	6.3			5,510	1230	3,350	63	244	4	248	
	2.5			3,000	380	4,200	14	75	7	82	
	0.9			345	40	180	1	4	1	5	
				262,455	45,590	112,870	2,134	10,829	94	10,923	
(NO$_3$) (NO$_2$)	7.8	120		2,840,000	800,000	910,000	160,000	260,000	30,000	295,000	4,550,000
	3.1			315,000	135,900	270,000	6,300	27,100	900	27,900	
	6.1			23,800	10,200	15,300	3,400	10,200	340	10,500	
	6.1			571,000	286,000	286,000	5,710	34,200	571	34,800	
				909,800	431,200	571,300	15,410	71,500	1811	73,200	
				4,660,955	1,376,790	2,116,370	185,594	373,899	41,225	419,933	5,564,800
			USE	4,660,000	1,400,000	2,120,000	186,000	374,000	41,200	420,000	5,560,000

B

ultimate demand of the BOD and to determine the reaction rate, a long term BOD was run. This is discussed in Part 4 of this report.

5. Total Input Loads to Basin

In order to compute total loads to the basin the mean daily flows were obtained from the continuous flow gauge recording of the streams. The overflow volumes were computed from the recorded flow data on a mean monthly basis. Volumes of both streams and overflows were computed in acre-feet. Computations of the loads were based on sampling data using median values. The mean daily discharge includes low flow as well as the storm runoff. During the sampling period, there were no unusual runoffs since rainfall was within one standard deviation of the monthly mean rainfall over the 71 year period of record.

The 30 day loads from the streams (Item 1) and combined sewer overflows (Item 2) were computed and are shown in Table 2-6.

The Easterly Wastewater Treatment Plant effluent load is also shown on Table 2-6 as Item 3. The quality parameter concentrations used are those of 1966 and 1967 monthly averages. The flow quantity used is the projected flow for the year 2000, which is 151 mgd. Presently, the mean daily flow averages 120-125 mgd. It is not expected that the concentrations will change substantially, unless tertiary treatment is added. The effluent from the Easterly plant constitutes the largest percentage of flow and pollutional load. For example, Easterly contributes 54 percent of the total input COD, 48 percent of the BOD and 39 percent of the suspended solids.

The sum of the loads for Items 1, 2 and 3 represents the average 30-day load. This sum does not include any load from the selected design storm occurring during the 30-day period, which would impose an additional load. The design storm selected to represent this additional pollution load was the runoff resulting from a consecutive 3-day storm which would have a frequency of occurrence of once in three years, and is identified in the table as Item 4. The concentration of the quality parameters for this storm flow was estimated from high flow data during storms from both a combined sewered area and separate sewered area. The excess Easterly Plant flow occurring during the storm was assumed to have the same concentrations as flow from a combined sewer area. These concentrations are somewhat lower than those found in a detailed study of combined sewer overflow on the West side of Cleveland since the intensity of the design storm is greater, and the additional dilution water will tend to lower the concentrations. Table 2-6 shows the estimated concentration and loads for the design storm as well as the total loads to the basin for the 30-day detention period. The figures given in Table 2-6 were used for the preliminary designs described in Parts 5 and 6.

PART 3

REVIEW OF RELATED PROJECTS

The improvement of polluted waters through the use of various types of storage impoundments and stabilization basins has been studied for many years, and is well documented in the literature. In this section, a review is presented of some of the past experience with related projects bearing on the present study.

In 1931 Imhoff(3) reported on the use of eight large impounding reservoirs for increasing the period of self-purification of the polluted water in the Ruhr River. Aggregate capacity of the reservoirs was about one billion cubic feet, and their construction was undertaken as a means of increasing the dissolved oxygen content of the river. Although these impoundments were not sufficiently effective to substitute for biological treatment as was originally proposed, they did accomplish substantial improvement in water quality, and remain a feature of the Ruhr pollution abatement system.

In England, the Metropolitan Water Board has used storage reservoirs for many years for pre-treatment of polluted Thames River and Lee River waters, prior to filtration for the water supply of London. A thorough and continuing study is made of the changes in water quality which come about in these large impoundments, which range from 1,749 to 6,679 acres in area, and from 29 feet to 57 feet in depth. Examination of the operating data of the Queen Mary Reservoir(4) illustrates the water quality changes taking place in the storage reservoirs. This is the principal flow-through reservoir; it is about 707 acres in area and about 39 feet deep, roughly circular in shape, and has a theoretical retention time of about 45 days at average flow rates; the only artificial treatment carried on in the impoundment is occasional dosage of copper salts for algae control. Results of operation of this basin over the period March 1961 to March 1965 show that the average reductions of various constituents was as follows:

Constituent	Reduction, %
BOD	65
Turbidity	79
Ammonia Nitrogen	37
Total Nitrogen	16
Total Phosphate	14
E. Coli	98

In the United States, a classic example of water quality improvement by impoundment was the use of reclaimed water at Chanute, Kansas. In 1956, as the result of a severe drought, the City of Chanute commenced re-cycling sewage treatment plant effluent through a stabilization pond prior to treatment in the municipal water purification works for re-use as public water supply. The stabilization pond received an average flow of about 1.25 mgd, and the mean retention period in the stabilization pond was 17 days. It was reported by Metzler, Culp, et al(5) that the impoundment served effectively as a waste stabilization basin. Reductions in waste loads achieved in the stabilization basin alone were reported as follows:

Constituent	Reduction, %
BOD	75
COD	75
Ammonia Nitrogen	55
Total Nitrogen	55
ABS	50
Total Phosphate	55

Fall(6) reported operation of a pilot plant retention pond receiving effluent from the high rate activated sludge treatment plant at Peoria. In a 10-month period of operation, about 0.65 mgd of plant effluent passed through the basin with a retention period of approximately 2 days. It was reported that the retention basin effected removals as follows:

Constituent	Reduction, %
BOD	27
Suspended Solids	70
Volatile Suspended Solids	70
COD	26

Based on the successful results of this pilot plant, full scale effluent retention basins were built at Peoria, which have been in operation since July, 1966. These basins total 22 acres in area, and vary in depth from about 11 to 16 feet. Operating results over the period of October, 1966 to August, 1967 showed a mean detention period of about 4 days at an average flow of 20.5 mgd. Although the data show substantial variations in monthly average results, the pond has been successful in improving overall plant performance; removals of BOD in the pond are in the range of 30 to 40 percent, and suspended solids removals are in the range of 50 to 70 percent. Overall plant performance has been improved from the range of 86-89 percent BOD removal to the range of 91-94 percent.

Some algae growths occur; free-floating clumps accumulating near the outlet are removed periodically to prevent odors. It is believed that sedimentation is the principal removal mechanism, coupled with anaerobic digestion of the deposited organic solids. There has been no significant odor problem as long as the small amount of floating material is not allowed to accumulate.

A similar effluent stabilization basin in England has been described by Taylor(7). At the sewage treatment works at Rye Meads, Hertfordshire, treated effluent was discharged to a series of lagoons for tertiary treatment. Three lagoons were operated in series, receiving a controlled flow of 1.5 mgd and providing an average overall detention period of about 17 days. The treatment plant is a secondary plant using trickling filters. In 29 months of operation (from May 1962 to September 1964), the stabilization lagoons showed substantial treatment effectiveness. Removal of BOD_5 in the basin averaged 70 percent; removal of suspended solids averaged 84 percent; dissolved oxygen was increased in the lagoons from an average of 39 percent of saturation in the influent to 89 percent of saturation in the effluent; and it was noted that the lagoons provided a valuable buffering action in preventing sudden adverse fluctuations in quality from reaching the river. It was stated that much of the improvement in quality was due to sedimentation and stabilization of soluble organic matter. At times, algae growths occurred and were responsible for periodic increase in values of effluent BOD. No nuisance problems were reported.

Rigby has described the operation of an effluent stabilization basin in Columbus, Ohio(8). Treated effluent from an activated sludge plant was retained for an average period of 5.3 days in an effluent stabilization tank 8 feet deep. Over a 9-week test period, the average removal of BOD in the basin was found to be 55 percent, and suspended solids removals averaged 64 percent. DO levels ranged from 1.8 to 7.2 mg/l in the basin. Other parameters were not observed. Fish were able to live in the basin, and it was stated that sludge accumulation after 19 weeks operation was negligible. It was stated that the principal removal mechanism was believed to be sedimentation followed by digestion of the deposited solids.

In Helsinki, Finland, a large storage impoundment is used to improve quality of a polluted surface water prior to treatment for water supply. Nagler(9) has described the operation of this reservoir, which contains 1.3 billion gallons of water, has a surface area of 120 acres and a depth of 52 feet. Water from a polluted river is pumped into the reservoir in such a manner as to induce continuous jet mixing of the impoundment. Mean detention period is 21 to 28 days. Data on the reservoir operation for 1965 showed reductions of 30 percent in ammonia nitrogen; 53 percent albuminoid nitrogen; 89 percent in B Coli; 92 percent in E. Coli; and over 90 percent in bacteria counts at 20°C and 37°C. BOD was not reported. Due to the continuous mixing and circulation employed, sedimentation is not effective, and no reductions in solids are obtained. It is stated that, although algae are present in the river water, there has been little or no blue-green algae growth in the reservoir, which is attributed to the continual motion of the water. A fish population has been established, and in four years of operation, the impoundment has become increasingly effective in water quality improvement.

In all of the cases cited above, the stabilization basins were fairly deep, ranging from 8 feet to 57 feet in depth. Most of these were subject to thermal stratification, and it is probable that intimate mixing throughout the basin depth does not occur except in cases of seasonal turnover or deliberate mixing. An example of a different type of impoundment is found at the Santee Recreation Project in California.(10) In this extensively studied water reclamation scheme, effluent from a 2 mgd secondary treatment plant is discharged into an effluent stabilization and oxidation pond, prior to chlorination, percolation and further treatment for use in recreational lakes. The stabilization pond has a surface area of 16.3 acres, contains about 24 million gallons, and varies in depth from about 10 feet at the inlet end to 2.5 feet at the outlet. From 1962 to 1968, the high rate treatment plant has been inadequate to produce a good effluent; the plant effluent discharged into the stabilization pond has averaged 62 mg/l of suspended solids, with a range of 23 to 314 mg/l; volatile solids in the pond input averaged 54 mg/l. BOD of the pond influent varied over the range 14 to 120 mg/l, with an average of 25 mg/l. In addition, during part of this period, digester supernatant liquor was discharged directly into the pond. The wide range of plant effluent indicates the generally unstable performance of the overloaded treatment plant.

New and enlarged treatment facilities are under construction, which will undoubtedly produce better results and more uniform loadings to the tertiary treatment processes.

In general, the Santee stabilization pond has exhibited performance of an overloaded oxidation pond. BOD and suspended solids removals were poor, and irregular. Ratios of carbon to nitrogen and phosphorous were unfavorable for good biological activity, being unbalanced toward nitrogen and phosphorous. Algae growth is heavy at times, and a high level of photosynthetic activity generally takes place during the period of June through August. Stratification occurs in the pond due to the hot summer climate, but because the pond is relatively shallow, high winds which often occur during the late summer and early fall can easily overturn the pond. Overturning releases large quantities of ammonia nitrogen, phosphorous and organic solids stored in the bottom sludge, which may cause rapid algae die-off followed by intense algal blooms, sometimes of other species. During the intense summer period of photosynthesis, pH rises as plants take up carbon dioxide, and ammonia, phosphates and alkalinity are removed from the water. Algae die-off sometimes causes odor problems, which have been partially controlled by installing diffused air aeration in a portion of the influent end of the pond.

The stabilization pond was effective in reducing bacterial and virus concentrations in the plant effluent. Total coliform, fecal coliform and fecal streptococci removals ranged between 96 and 99 percent in the stabilization pond.

Operation of the Santee stabilization pond illustrates the fact that while plant nutrients, particularly phosphorous and nitrogen, may be removed from the liquid being treated, they are stored within the pond in algal cell material and in the precipitated sludge deposits. In order to prevent gradual nutrient build-up with resulting algae blooms and nutrient loss into the receiving water, either the deposited sludge solids or the algae should be periodically removed for permanent disposal, or sealed from contact with the overlying water. In the new treatment plant at Santee, nutrients will be removed in a tertiary treatment process prior to discharge into the stabilization pond.

Although the Santee effluent stabilization pond has been successful from the standpoint of its place in the overall water reclamation scheme, it would not have been satisfactory as the sole means of effluent stabilization.

Other extensive studies of the effects of impoundment upon water quality are reported in the literature.(11)(12)(13) In a discussion of this subject by Love,(14) the favorable effects of storage on water quality are summarized as: reduction of turbidity, silica, color and coliform bacteria; evening out of sharp variation in dissolved minerals, hardness, pH and alkalinity; reductions in temperature; sedimentation of solids; dilution of wastes; and nutrient uptake by algae. Unfavorable effects listed include: increased algae growths; reduction in dissolved oxygen in deeper parts of the reservoir; increase in carbon dioxide and sometimes iron, manganese, and alkalinity near the bottom; increase in dissolved solids by evaporation;

and reduction in temperature. Some of the effects mentioned would not apply to impoundments of short detention time, or those characterized by continuous outflow.

Connell and Berg(15) have reported on the re-use of municipal wastewater for industrial water supply purposes. Their survey of 1959 showed that in 19 cases of effluent reclamation, all but one plant practiced storage of effluent prior to further treatment and re-use, for the purpose of stabilization and equalization of load.

In Mexico City effluents from conventional secondary treatment plants are discharged into lakes located in the city parks, for purposes of aesthetic and recreational enjoyment. These lakes provide stabilization of the effluent, and have been reported free of nuisances. Boating is permitted although swimming is prohibited.

In addition to the references cited, an extensive literature exists on the subject of waste treatment by oxidation ponds. Although some of the features and processes of an oxidation pond are similar to those expected in the proposed stabilization basin, the differences are perhaps more significant than the similarities. Oxidation ponds depend heavily upon the photosynthetic activity of green plants, and their successful operation requires the maintenance of aerobic conditions throughout their depth. Most ponds discharge more or less stabilized solids in the effluent, and would quickly fill up if this were not the case. The characteristics and flow rates of the input loads in the case of most oxidation ponds are those of raw sewage.

On the other hand, the stabilization basin input load consists largely of residual loads contained in a treated effluent. For these reasons, as well as limitation of space, experience in operation of oxidation ponds is not fully applicable to the proposed stabilization basin, and will not be considered in detail here.

From the examples cited, several conclusions may be drawn regarding impoundment of treated effluents and polluted waters:

1. Such impoundments improve the quality of the waters so treated.
2. The principal improvements are brought about by:
   - (a) Stabilization of the wastes.
   - (b) Removal of suspended solids by sedimentation (including finely divided activated sludge floc).
   - (c) Reduction of BOD and COD by biological oxidation of organic matter through both bacterial and algal systems.
   - (d) Major reduction in total and fecal coliforms, fecal streptococci, and viruses.
   - (e) A buffering action which evens out peaks in flow rates, pH, and concentration of chemical constituents.
   - (f) Removal of phosphorous and nitrogen by algae uptake. (For this removal to be effective, some means of permanently retaining these nutrients must be included.)
   - (g) Stabilization of the settled sludges through anaerobic decomposition.
3. Stabilization basins may exhibit some undesirable effects, such as:
   - (a) Excessive algae growth, accompanied by discharge of algae mass in the effluent.

(b) Accumulation and die-off of algae which may produce odor or other nuisance conditions.

(c) If excessively loaded and anaerobic, the basin can produce odors, surface scum, and other nuisances.

(d) Build-up of nutrients, iron, manganese and other materials which may adversely affect water quality.

The success of the proposed stabilization basin depends upon promoting the beneficial processes and controlling the detrimental ones.

PART 4

GENERAL DESIGN CONSIDERATIONS

A large stabilization basin such as that proposed constitues an ecological environment in which many chemical, physical, and biological processes are at work. The success of the basin in achieving its objective in pollution abatement depends upon promoting those processes favorable to our objectives, and controlling those which are detrimental. This objective is simpler to state than to carry out, because we know little about the detailed mechanisms operating in some of the biological processes, and also because various desirable processes require different environmental conditions than others and cannot be simultaneously optimized.

In this part of the report, the considerations bearing upon the basin design will be discussed in general terms; in Part 5 the detailed design criteria are presented.

1. Sedimentation

The greatest single input load of suspended solids to the proposed basin is the Easterly Plant effluent load, estimated at about 30,000 pounds per day (in the design year 2000). This is about 43 percent of the total suspended solids input to the basin. These solids are composed principally of finely divided activated sludge mass which has escaped the final settling process, and is estimated to contain about 85 percent volatile matter. This solid matter carries a substantial portion of the BOD load, since much of this sludge mass is probably biological cell matter undergoing respiration.

Optimum removal of this type of material requires quiescent settling with low surface settling rates and long detention time.

2. Biological oxidation

Biological oxidation is taking place in the Easterly plant effluent when it is discharged into the basin and will probably be occurring to a limited extent in the combined sewer and stream flow inputs. It is desired to carry on bio-oxidation to the greatest extent feasible.

It is probable that bio-oxidation of the treatment plant effluent is occurring in the declining growth phase or in the endogenous respiration phase. The food/mass ratio is low, and the oxidation rate is correspondingly low. The oxidation rate is a measure of the time required for the microorganisms to reduce the organic materials in the waste, and is generally referred to as the "K-rate". For preliminary evaluation of the oxidation rate, a long-term K-rate study was performed on Easterly effluent samples, by the Elmore technique. The resulting BOD curve is plotted in Figure 4-1, from which the K-rate was determined to be 0.031. This test indicates that about 30 days will be required to satisfy 90 percent of the ultimate BOD.

Reaction rates of the BOD contained in the raw sewage of the combined sewer overflow and the polluted streams will be higher, and has been estimated after study of a number of laboratory K-rate determinations at about 0.10, indicating that oxidation periods on the order of 20 days might be required to satisfy the carbonaceous BOD of this waste. It must be appreciated that these estimates are based on relatively few catch samples, and are to be viewed as first approximations only.

These considerations indicate that an overall BOD removal of 70 to 85 percent is possible with detention periods on the order of 30 days. Optimum conditions for biological oxidation necessitate maintenance of adequate levels of dissolved oxygen, good mixing to assure contact between the microorganisms and substrate, maintenance of reasonably uniform temperature (as high as feasible), and provision of a large water mass to minimize the effects of possible toxic discharges or slug loads.

3. Removal of Coliform and Other Bacteria

High percentage removals of total and fecal coliforms, fecal streptococci and viral organisms may be expected on the basis of past experience with similar impoundments. Experience cited in Part 3 of this report indicates that die-away of these indicator organisms may be expected to exceed 95 percent in a 30-day impoundment of a treated plant effluent without chlorination.

In order to confirm this indication and to evaluate the rate of die-away, we have conducted laboratory tests in which effluent from the Easterly Plant was mixed with Lake Erie water in various dilutions, and maintained under aerobic conditions for periods up to 9 days. Figure 4-2 shows curves plotting counts of total coliform organisms, fecal coliforms and fecal streptococci against time. The curves indicate that 95 percent die-away occurs under these conditions in about 7 days.

These tests were made without addition of fresh sewage or other food during the test period.

However, it may be expected that re-growth of the coliform population may take place to some degree when the input streams expose fresh food materials. The re-growth phenomenon has been studied by many investigators. In a recent study, Eliassen(16) found that within 30 hours after discharge of combined sewer overflow into tidal basin water, the organic food in the overflow is sufficient to promote rapid growth of the coliform population to peak values ranging from 10 to 40 times the original number at discharge. Chlorination of the sewage before discharge reduced the numbers of organisms, but after-growth still occurred, and peak values on the order of 20 times the original number were observed, with coliform counts of 400,000 to 500,000 organisms/100 ml occurring at the peak. Following the after-growth peak, die-away follows, and in many cases more than five days detention was required to reduce the numbers of coliform organisms to the original value at discharge.

Optimum bacterial reductions require removal of the organic food materials (by bio-oxidation) and long detention period. Of course, chlorination could be applied in the stabilization basin, but after-growth would still take place after the basin effluent was discharged into Lake Erie if sufficient organic material were available as food.

In a basin of more than 20-day detention, high bacterial reductions may be expected.

4. Nutrient Removal and Algae

Nutrient removal was not considered a primary objective of the proposed stabilization basin. There is little doubt that a high level of phosphate removal can be achieved in special treatment processes in a plant designed for this purpose, and that this can probably be done more economically in a high rate plant process than in a stabilization basin. Although this may be the case, any nutrient removal accomplished in the basin is of added benefit to the primary objectives.

It has been assumed in the preparation of this study that percentage removal of phosphates and nitrogen in the Easterly Plant process will remain approximately at their present levels (i.e., tertiary treatment processes for removal of nutrients have not been instituted). On this assumption, it is estimated that the total phosphorous input in the year 2000 will approximate 6,200 pounds per day, and the total nitrogen load will be about 14,000 pounds per day. These quantities are far in excess of the necessary amounts of primary nutrients in comparison with the available carbon (as represented by BOD), which is to say that the basin influent is nutrient rich. In the absence of toxicity or other inhibitory factors, substantial algae growth may be expected to take place in the basin.

In order to evaluate the effects of algal growth, and to project possible problems of algae control, it was desired to have the benefit of the advice of an expert algologist and aquatic biologist. For this purpose, the services of George P. Fitzgerald of the Water Chemistry Laboratory, University of

Wisconsin were retained. Dr. Fitzgerald's comments and projections are discussed in the section of this report on basin management and operation in Part 7.

Without presenting here the reasoning which justifies the conclusion, it is estimated that under summer conditions, about 50 percent removal of both nitrogen and phosphorous will take place through algae growth and precipitation, that removal of nutrients would be optimum with pH in the range 9.0 or greater, and that mixing of the basin to expose more of the water to light would promote algal growth. In a mixed, aerobic basin, production of algae could probably exceed 400 pounds per acre per week.

Nutrient removal would be accomplished partly by biological uptake by algae and other biota, and partly by natural mineral precipitation. Nutrients removed by these processes will be deposited on the bottom by sedimentation of dead cell material and precipitates, along with other suspended solids, and it is recognized that under anaerobic conditions, much of the nutrient material can be solublized and released to the water if the bottom sediments are not sealed or removed.

5. Other Biological Life

If the stabilization basin is maintained in an aerobic condition, it may be expected that a rather broad and varied biological system would be established. In the Thames impoundments and elsewhere in similar cases, a rich and varied population of aquatic plants and animals is found. Although it is not possible to predict accurately the characteristics of the ecological system of the proposed basin, a wide variety of phytoplankton, zooplankton, various insects and a variety of fish may be expected. Establishment of a productive and varied biology will be beneficial, since the food chain on which it subsists is based upon the organic waste materials which we wish to remove.

However, it may be necessary to exercise controls to prevent establishment of an over-productive biology accompanied by nuisance conditions and undesirable discharges to the lake.

Establishment and maintenance of a good biological system requires an aerobic basin, (at least in a significant upper layer), absence of toxicity, and a relatively constant food input. A large basin volume, with its attendant capacity to absorb shock loadings, is also desirable.

6. Structural Design

The proposed basin consists basically of a wall enclosing a portion of the inshore waters of Lake Erie. The basin wall, which would be exposed to the full wave and ice forces of Lake Erie, must be designed to resist these forces in the same way as a harbor breakwall.

It is desirable that the basin wall be substantially watertight, although a limited amount of diffusion of water through the wall would not be a serious defect.

At the site available, water depth ranges from 10 to 45 feet, with the deep water at the northerly or lakeward side of the basin. To obtain a feasible structural design with these water depths, it is assumed that the static water level inside the basin is substantially the same as that in the lake, and that the basin cannot be dewatered. Differential water levels due to wave or ice action will occur, and will be taken into account in the structural design of the wall.

7. Miscellaneous Design Considerations

Other factors to be considered in design include the effects of the basin on lake navigation, features affecting shoreline erosion and beach protection, recreational aspects of the project, and aesthetic considerations of the basin structure and operation. These will be discussed in detail in Part 5 and Part 8.

PART 5

PRELIMINARY DESIGN - STABILIZATION BASIN

1. General Design Features of the Basin

The general design considerations noted in Part 4 indicate that all of the desirable functions of the proposed stabilization basin cannot be achieved under the same physical conditions. For example, good sedimentation requires quiescent conditions, while maintenance of uniform dissolved oxygen requires mixing. Similarly, biological oxidation requires both dissolved oxygen and circulation, while retention of the deposited solids depends on avoiding circulation of water, at least near the bottom.

To meet these varying needs, it is proposed to subdivide the stabilization basin into three zones: a first stage zone in which the basin contents are mixed and aerobic conditions are maintained; a second stage which is a quiescent sedimentation and clarification zone; and a third zone where the treated water is re-aerated and chlorinated prior to discharge into Lake Erie. It is contemplated that the three zones of the basin would operate generally as follows:

(a) First Stage, Aerobic zone. The first stage zone would be a biological oxidation stage. Incoming loads would be thoroughly mixed with the basin contents and the basin would be artificially circulated and aerated to maintain satisfactory dissolved oxygen levels throughout this part of the basin. Oxidation and stabilization of the organic loads would take place, and this zone would be an active growth environment for algae. The provision of aerobic conditions would minimize possibilities for odor nuisances, and agitation would tend to prevent the accumulation of surface scum. Long term biological oxidation would probably proceed into the endogenous respiration stage, and organic and ammonia nitrogen would be stabilized to nitrate. Detention period in the first stage zone should be long enough to carry out a high degree of biological stabilization, permit nutrient uptake by algae, extend beyond the re-growth phase of coliforms, absorb fluctuations in load, and provide a suitable environment for aquatic life. It must also be recognized that this zone should be large enough to allow for hydraulic inefficiency, so that the mean effective detention period is realized. Based on evaluation of other stabilization basins and on the low oxidation rates expected, detention period in this zone should be about 15 to 20 days.

(b) Second Stage, Settling Zone. The second stage zone would not be mixed or artificially aerated. Sedimentation of suspended solids would take place under reasonably quiescent conditions. It is expected that the upper 8 to 10 feet of this zone would remain aerobic through surface re-aeration and release of oxygen by algae; the lower 30 to 35 feet would be low in dissolved oxygen, with the deeper layers being anaerobic. The deposited solids would form a sludge blanket at the bottom, which would undergo some anaerobic decomposition. Some of the decomposition products would be released to the overlying layer of water, but without mixing in this deep impoundment, the transfer to effluent water should not be great. Gaseous decomposition products would rise through an aerobic layer before release to the atmosphere, which has minimized possible odors in similar impoundments. Most of the deposited solids would be retained in this zone. It is expected that the detention period in the second stage zone should be about 10 to 15 days to achieve the desired removals with adequate provision for agitation from wind and wave conditions and short circuiting.

(c) Third Stage, Effluent Re-aeration and Chlorination Zone. Prior to discharge into Lake Erie, it would be desirable to re-aerate the effluent water to avoid imposing a localized oxygen demand on the lake. This re-aeration period would be short, and the re-aeration devices would be located near the outlet ports. Detention period in the third stage zone would be 1 or 2 hours at peak flow rates. This zone would also provide the contact period for chlorination of the basin effluent for disinfection, if this is found necessary. Water quality criteria for Lake Erie require that average total coliform organisms not exceed 1,000/100 ml for recreational waters or 5,000/100 ml for public water supply. It is possible that these standards could be met without chlorination of effluent, but facilities should be provided for disinfection if coliform counts are higher.

Physical separation of the three zones by construction of rigid walls would be very costly and is not necessary. It is proposed that flexible baffle curtain walls be used for this purpose. The preliminary curtain design is based upon the provision for a heavy neoprene impregnated fabric, supported by pontoon floats, and held in place by cables anchored to the bottom. Fabrics of this type are available with tensile strengths up to 6,000 pounds per lineal foot, and they are resistant to chemical attack and ultraviolet radiation.

Separation of the third stage zone would require more rigid construction, since it is desired to control flow into this zone from various depths through ports. The purpose of depth selection is to avoid discharging surface waters which could contain larger concentrations of plankton, as well as bottom waters which would carry deposited solids out of the impoundment. Although the rigid baffle wall for the third stage zone would be more costly, it would be short in comparison with the baffle separating the first and second stage zones. It is proposed to construct the third stage zone baffle wall of steel sheet piling, braced to the outer basin wall.

2. Required Size of the Basin

To carry out the intended functions, the effective detention time required for both stages of treatment should total about 30 days. It should be appreciated that in a large impoundment located in the open lake at lake water level, ample allowances must be made for interference of wave and wind action. As will be explained in a subsequent section, it is uneconomical to construct the basin walls of sufficient height to prevent overtopping during mild storms. Thus, the effects of displacement and agitation caused by waves and wind must be compensated for by an adequate safety factor. In addition, the characteristics of inflow from land runoff are highly variable. The basin must have sufficient volume to absorb high fluctuations in flow input. Volume should also be allocated to compensate for possible increases in runoff rate from the service area, as a higher degree or urbanization increases the impervious area percentage. Finally, a large, undivided basin is inherently subject to some degree of short-circuiting of flow thereby reducing its hydraulic efficiency as a sedimentation tank. Hydraulic efficiency can be improved by construction of interior subdivision walls, but preliminary calculations show that the cost of subdivision is greater than the provision of additional area to compensate for the inefficiency.

After consideration of these and other factors, the preliminary selection of the basin volume was based on the following criteria:

(a) Provide a detention period of 30 days for the base average flow from the Easterly Plant and the surface streams, plus the storm runoff from the combined sewer overflows and streams due to a 3 consecutive day rainfall of magnitude expected to occur once in 3 years. This total input volume as shown in Table 5-1 is about 24,800 acre-feet.

(b) Volume displaced by wave action has been estimated at about 2,100 acre-feet during a 30-day period. In addition, an allowance is made for increase in runoff and hydraulic inefficiency of about 10 percent.

Based upon these criteria, the required basin volume is calculated as follows:

Base flow volume	24,800 acre-feet
Wave displacement	2,100
Allowance for increase and inefficiency (10%)	2,500
Total Volume	29,400
Say	30,000 acre-feet

At the proposed site of the basin, the mean water depth as taken from U.S. Lake Survey charts is approximately 36 feet. However, about 10 percent of the basin area must be located in shallower water to avoid the necessity for constructing excessively long outfall conduits. After allowing for this shallow portion, the weighted average depth of the entire basin is about 34 feet. Thus, the required area based on the weighted average depth is 910 acres; for purposes of the preliminary design, the basin area is assumed to be 900 acres.

3. Basin Loading and Anticipated Removals

The determination of the input loads presented in Part 2 are summarized in Table 5-1 for convenience in considering basin design. Average inflow volume is about 24,800 acre-feet per month, or approximately 270 mgd.

TABLE 5-1

SUMMARY OF BASIN LOADING

Constituent	Total Load Lbs./30 Days	Total Load Lbs./day, avg.
BOD	1,400,000	46,600
COD	4,660,000	153,300
Suspended Solids	2,120,000	70,600
Total Phosphorous (as P)	186,000	6,200
Total Nitrogen (as N)	420,000	1,400
Alkalinity (as $CaCO_3$)	5,560,000	185,300

The overall BOD load, based on a 900-acre basin, is about 50 lbs. BOD/acre/day. In a general review of various types of stabilization ponds, Oswald et al(17) have classified ponds as anaerobic, facultative and aerobic in terms of input loading, depth, and detention period. According to this classification the proposed basin falls into the category of a facultative pond, characteristically loaded at rates from 20 to 200 lbs. BOD/acre/day, and having detention periods of 20 to 30 days. Aerobic ponds (not artificially aerated or mixed) are those loaded at less than 20 lbs. BOD/acre/day, and have detention periods in excess of 60 days. Torpey, in studying pollution of estuaries(18), has stated that when BOD loadings are imposed on large bodies of water, dissolved oxygen levels drop to about 40% of saturation with BOD loadings of 10-15 lbs. BOD/acre/day, and will decrease to about 25% of saturation at loadings of 100 lbs. BOD/day. In the range of loadings contemplated herein (50 lbs./acre/day), Torpey observes that without artificial reaeration, degradation of the environment begins to take place, some biological forms migrate, algae and bacteria will predominate, and nitrogen will not be oxidized. Atmospheric reaeration is inadequate to maintain satisfactory DO levels.

These considerations, together with preliminary calculation of the probable oxygen supplied by surface reaeration, indicate that the input BOD loading is within acceptable limits, but some supplementary reaeration will be required to provide good BOD reduction and maintain satisfactory DO levels. The method of supplying this reaeration is discussed in a later section.

Solids loadings are relatively light for the size of the basin, but in view of the characteristics of the activated sludge floc and the expected production of algal solids, conservative sedimentation design is indicated. Loadings of phosphorous and nitrogen are substantial, and their ratio to carbon in the wastewater indicates a nutrient-rich input.

In predicting removals to be expected in the basin, it is helpful to review the experience in similar impoundments cited in Part 3, which is summarized in Table 5-2.

TABLE 5-2

WASTE REMOVALS CITED AT VARIOUS IMPOUNDMENTS

Location of Impoundment	Approximate Detention Period, Days	Reported Removals, % BOD	SS	Phosphorous	Nitrogen	Total Coliform
Queen Mary Res., Thames R.	45	65	79	14	16	98
Chanute, Kan.	17	75	NA	55	55	NA
Peoria, Ill.	4	35	60	NA	NA	NA
Rye Meads, Eng.	17	70	84	NA	NA	NA
Columbus, Ohio	5	55	64	NA	NA	NA
Helsinki, Fin.	25	NA	0	NA	53	89

Based on these data, on our previous BOD studies on the Easterly plant effluent, and on experience with biological oxidation elsewhere, we believe that the proposed stabilization basin could achieve the waste load reductions as follows:

TABLE 5-3

ESTIMATED WASTE LOAD REDUCTIONS*

Constituent	Waste-Load Reduction, %	Estimated average Reduction, %
BOD_5	67 to 73	70
SS	70 to 75	72
TS	60 to 65	62
COD	60 to 65	62
Total P	45 to 50	50
Total N	45 to 50	50

*Based on summertime conditions, and 15-20 days aeration, 10-15 days sedimentation-stabilization.

The frequency of overflow events will be reduced from those resulting from rainfall intensities of 0.02 inches per hour (which occur about 530 hours per year), to those resulting from rainfall intensities which have a recurrence interval greater than one year (which occur less than five hours per year). The reduction in volume will, of course, be great since of the average 17 inches of rainfall runoff, less than 1 inch is caused by rainfalls with a recurrence interval of more than a year. However, due to the large number of input points with various areas and times of concentration, the actual volume reduction would be difficult to predict without extensive hydrologic data.

The reduction in organic discharges, as measured by BOD and suspended solids to Lake Erie from the combined sewer system and land runoff have been computed at 67 percent for BOD and 69 percent for suspended solids.

4. Shape of the Basin

Since the cost of the stabilization basin is almost directly proportional to the length of the enclosure wall, optimum design requires that the perimeter be minimized. Ideally, therefore, the basin should be circular in plan. However, a circular plan does not satisfy other requirements. The main input loads are centered at two locations: the Easterly Plant site, which contains a major combined sewer overflow as well as the treated plant effluent; and the Dugway Brook area. Dugway Brook, Nine Mile Creek and Shaw Brook all enter the lake within a distance of 2,000 feet along shore, and together constitute the major part of the input flow from the western half of the service area. The Dugway Brook area thus is a natural and economical location for a collection point for input to the basin.

Therefore, in order to reduce the cost and complexity of the shoreline collection system, it is advantageous to design the basin to extend between the two major input locations, Dugway Brook, and the Easterly Plant site.

A second consideration regarding basin shape is the effect on the shoreline property. It is deemed impracticable and undesirable to use the shoreline itself as one wall of the impoundment, because nearly all of the lakeshore land in this area is owned by private owners who have the right to use the beach and to have access to Lake Erie. Condemnation of all of this land would be costly, and unnecessary. Therefore, the basin should be designed with its inner wall sufficiently far from shore to permit free access and use of the part of the lake enclosed between the basin and the shore. For the preliminary design, we have located the inner basin wall about 3,000 feet offshore for the main portion of the enclosure, with a section of the basin at each end extending closer to shore at the two input points. Thus, a sheltered bay would be created by the basin, in the shape of an elongated U, which would be about one mile long and two-thirds of a mile wide. It is intended that this bay provide an important secondary benefit of the project in the form of a recreational site, which will be discussed further in a later section of this report.

The shape of the basin enclosure and the location of the outer wall is dictated by a number of factors. It is necessary to equalize the detention period of the two main input flows, which requires that the outlet point be located somewhat toward the west of the basin center, since the input at the Easterly Plant is the larger flow. The most effective shape would be a narrow rectangle or trapezoid extending northward into the lake, but such a shape would place the outer wall more than 12,000 feet offshore. This location is impracticable, since the basin would obstruct the shipping lanes entering the east entrance to Cleveland harbor. This location would be costly as well, since the additional water depth would require more difficult and expensive wall construction

After considering these factors, a number of preliminary designs were prepared for basins of various shapes, and a selection was made of the shape that met most of the design requirements at a minimum perimeter length. The recommended basin shape, which encloses 900 acres and contains the required 30,000 acre-feet, is shown in Figure 5-1.

5. Inlet, Outlet and Access Features

At the eastern terminus of the proposed basin, three submerged inlet conduits conduct flows into the basin. The main Easterly combined sewer overflow conduit will conduct flow by gravity through two 120-inch submerged pipes. Treated effluent from the Easterly Plant will enter the basin through twin 108-inch pressure pipes. It will be necessary to lift the plant effluent to provide sufficient hydraulic head to enter the stabilization basin. The third input will be a single 120-inch pressure pipe delivering flow from the eastern shoreline collector system, from the Euclid Creek area. The basin will contain inlet baffle walls to utilize the input velocity for jet mixing of the inlet streams, and to dissipate the excess energy for reduction of short-circuiting.

At the western terminus of the basin, two inlet conduits are proposed. The western shoreline collection system will discharge flow to the basin through two 144-inch pipes; the second inlet conveys the Dugway, Shaw and Nine Mile flows into the basin by gravity through a double 120-inch conduit.

At both eastern and western input points, an access channel about 350 feet wide is provided to permit boats to enter Lake Erie from the enclosed bay. The Easterly terminus of the basin is connected to shore by a causeway carrying a two-lane road to provide access to the recreational facilities discussed in Part 8. The causeway would provide adequate vertical clearance for power boats, but sailboats would use the westerly entrance channel only.

The treated effluent from the basin would enter the lake through multiple submerged discharge ports, at a point about 8,000 feet from shore. To provide optimum dispersion and initial dilution of the effluent into the lake waters, detailed design studies will be required to determine the required number, diameter and depth of the ports. Preliminary studies indicate that initial dilution on the order of 30 to 50 is feasible, depending upon the amount of hydraulic energy available. The effluent ports will draw water from a depth of about 10 feet below the surface, to avoid discharging algae which could tend to concentrate in the surface layers.

Access to the interior of the basin is provided by an entrance channel to be located on the eastern face of the basin. This access channel will be used by the work boat used for basin maintenance, and by the dredges used for periodic sludge removal. The channel will normally be closed by a floating pontoon gate to prevent private boats from entering the basin.

6. Aeration and Mixing

The destratification, mixing and aeration of large impoundments has been intensively investigated within recent years. Thermal stratification tends to bring about various undesirable conditions in deep impoundments associated with depletion of dissolved oxygen, and numerous researchers have been working to develop methods of destratification for improvement of the dissolved oxygen content, to provide more uniform quality of water throughout the impoundment, and to reduce temperature gradients.

The most notable efforts in this area in the United States in recent years has been the work done by a group headed by J.M. Symons of the Federal Water Pollution Control Administration in Cincinnati, Ohio. This work has extended over several years, involving a number of lakes and impoundments in Kentucky, and has been reported extensively in the literature.(19)(20) Dr. Symons' group found that large impoundments can successfully be destratified and aerated by both mechanical pumping and diffused air mixing. The controlling dynamic criteria have been investigated, and an engineering design approach has been defined. In general, it was found that the energy input required to mix and aerate an impoundment is related to the temperature differentials existing between the thermal layers, and this energy input has been quantitized by the basin stability concept. It was found that a surprisingly small power input is required to successfully mix and aerate these impoundments. Dr. Symons concludes from his work, and that of other investigators, that the energy input required ranges from 0.7 to 4.9 KWH/acre-ft., averaging about 2 KWH/acre-ft. Oxygenation capacity of the mixing devices was found to average about 1.3 lbs. O_2/KWH as measured by net change in oxygen balance in the impoundment over a measured mixing period.

Similar work in Germany has been reported by Bernhardt(21). At Siegburg, the Wahnbach Reservoir has been successfully destratified and aerated by release of compressed air, as well as low lift mechanical pumping and by a combination of the two. This reservoir contains 33,740 acre-feet, and has an average depth of 63 feet. It was found that the energy input required was 3.1 KWH/acre-ft. of volume, and that the oxygenation capacity was 2.1 lbs. O_2/KWH for the diffused air mixing.

The large Thames River impoundments previously referred to have been successfully aerated by both mechanical pumping and jet mixing, and other examples are reported in the literature which are essentially in agreement with those cited.

Re-aeration of rivers and other polluted waters has also been studied, but to a lesser extent than impoundments. Various techniques have been employed for this purpose, including mechanical aerators, diffusion of compressed air, aspirators, and pressure injection. Mechanical surface aerators were employed in research work for the Chicago Sanitary District as reported by Kaplovski(22) and tests have also been conducted at Minneapolis and on the Ohio River.

Results of these various field tests indicate oxygen transfer rates ranging from 0.5 to 4.5 lbs. O_2/KWH, averaging 1.7 lbs. O_2/KWH. Since these tests were conducted on polluted waters having an oxygen uptake, they are expected to be reasonably similar to the requirements in the proposed stabilization basin.

Precise determination of the number and size of reaeration and mixing units required for the aerobic first stage zone must await the detailed design, but for preliminary purposes, it may be assumed that an energy input of about 2.5 KWH/acre-ft. will be required for full reduction of the basin thermal stability. On this basis, a total power input of about 80 KW will be required to reduce the basin stability in 30 days. It is assumed that three 40 horsepower mixer-aerators will be employed. For supplementary oxygenation and mixing of the enclosed bay, a fourth unit has been included in the estimates of cost.

7. Structural Design of the Basin Wall

The study has included preliminary structural designs of various types and configurations of the enclosure wall for the proposed basin. These structural design studies have been based on rather meager data presently available concerning the subsoil and foundation conditions at the basin site, and on limited desk-top evaluation of the dynamic forces to be dealt with. No subsurface exploration, soil testing or hydraulic model studies were included in the scope of this preliminary study.

The preliminary structural design work included computation of the design wave forces expected, design of the wall for stability and soil bearing loads, consideration of alternative methods and materials for wall construction, consideration of effect of various freeboard heights, and applicability of various construction methods. A review of the structural design procedures and findings is presented in Appendix B. The results of the structural design study and the recommended preliminary design criteria are summarized as follows:

Type of wall - Cellular steel sheet-piling cofferdam, filled with sand or crushed stone. (Selection of fill material subject to determination in final design.)

Cell diameter - 67 feet, approximately.

Cell spacing - 72 feet, center to center.

Total basin perimeter - 30,000 feet, approximately.

Total number of cells required - 418.

Design water elevation - 573.3.

Freeboard, (top of wall above DWL) - 10 feet.

Foundation bearing load - 6,280 pounds per square foot.

Overturning moment - 1,562 ft.-kips.

Design wave: Height - 17 feet
Length - 128 feet
Period - 5 seconds
Recurrence interval - 25 years

Expected frequency of overtopping - 6.2 times per year.

PART 6

PRELIMINARY DESIGN
SHORELINE COLLECTION SYSTEM

This part of the report is concerned with the collection system required to intercept the combined sewer overflows and surface streams and to convey these flows to the offshore stabilization basin.

The data presented in Part 2 were used for the preliminary design. The collection system cannot be practicably designed to intercept the runoff from intense storms of low frequency of occurrence. To evaluate the flow rates to be expected, runoff hydrographs were prepared for each surface stream for the 1, 3, and 5-year storms. The critical criteria for the collection system is the peak rate of runoff to be expected from the design storm, while the critical criteria for the basin is the total volume of runoff expected in a 30-day detention period.

After detailed consideration of the hydrographs and preliminary routing, the one-year rainfall was adopted as the design storm for the collection system. The peak discharges for each input point resulting from the one-year storm are given in Table 2-5.

Site topography and location of the load inputs led to the selection of two general area systems. The flows occurring between East 72nd Street and Nine Mile Creek would be collected and conveyed to the offshore basin at the west end of the basin near Eddy Road and Dugway Brook. This has been termed the West Collection System and is shown on Figure 6-1. The flows from Green Creek east to Neff Road would be collected and conveyed to the basin at a point near the Easterly Plant. This has been termed the East Collection System and is shown on Figure 6-2.

1. The West Collection System

The West Collection System has two parts. One part would collect the flow from E. 79th Street, Doan Brook and E. 88th Street. The flow from these input points would be discharged to a holding tank to reduce the peak discharge rate, screened and pumped to a point near Dugway Brook. Dugway Brook flow would join this flow after being screened, and would be conveyed by a gravity system into the west end of the basin, through a submerged conduit. The second part of this system is the gravity collector from Nine Mile Creek and Shaw Brook. The flow would be screened and then conveyed to the basin through another submerged pipe.

The Easterly Plant overflow is conveyed through a submerged gravity conduit directly from the plant to the basin. The plant effluent will reach the basin through a submerged conduit also. The flow will have to be pumped when the lake level is above 572.0 to prevent flooding of the final settling tanks. During the past 98 years the mean monthly lake level has been above Elevation 572 for 64 percent of the time.

A collection system which would not require any pumping was also investigated. The conduit cross-sections required to convey the peak flow, the low velocity due to the limited head available, and the long distance involved made the all-gravity system undesirable. In addition the cost of excavation and conduit was found to exceed the combination pressure-gravity system proposed by approximately 75 percent.

The cost estimate for the Easterly Plant effluent pumping station and force main, and the Easterly Plant overflow conduit, which convey this flow to the basin, are included in the cost estimate for the West Collection System in Part 9. However, these two conduits are shown in Figure 6-2 with the East Collection System.

2. The East Collection System

The East Collection System carries flow to the basin from Neff Road, Euclid Creek, East 156th Street and Green Creek. This collector would require pumping of the Euclid Creek and Neff Road flows and a second pumping station for East 156th Street and Green Creek. Holding basins would be provided at each pumping site to reduce the peak pumping requirements. Fig. 6-2 shows this system.

An alternate gravity system was investigated for the Euclid Creek to Green Creek inputs. The large flows from Euclid Creek required conduits of such large size that the cost was prohibitive. Open channels next to the shore were considered, but were also more expensive than the proposed system as well as being esthetically undesirable.

Additional details and routed hydrographs are given in Appendix C.

3. Location

Both collection systems would be constructed next to the shoreline. Preliminary investigation of the routes indicate little or no disturbance to beach area since these areas have low bluffs at the shoreline, and the rocky beach is not suitable for recreation or bathing. Preliminary test borings along the route indicate no problem with rock or other sub-surface difficulties.

Other locations back from the shore were also considered. Excavation methods such as deep cuts and tunneling were investigated along these other routes, and the proposed location and construction method were selected for the preliminary design work as most feasible. More extensive sub-surface investigations will be required in final design to confirm this finding.

PART 7

OPERATION, MAINTENANCE AND CONTROL

The success of any treatment process depends on the satisfactory operation and control of the facility. In the case of the proposed stabilization impoundment, these functions include routine operation and maintenance of the pumping stations and of the trash racks and chlorination facilities of the collection system; operation of the mixing and reaeration devices; control of algae and other biological life to prevent nuisance and enhance treatment; periodic disposal and control of sludge deposited in the basin; and the necessary surveillance and monitoring of the system.

1. Collection System

Routine operation and maintenance of the pumps and other mechanical equipment of the collection system does not require detailed discussion. It is contemplated that the collection system pumping stations will be automated, and that continuous attendance would not be required. Periodic removal of debris collected on the trash racks will be required; it is expected that this material will be trucked to the municipal refuse incinerator or sanitary landfill sites for disposal. Excess storm flows will be disinfected prior to discharge into Lake Erie by the application of sodium hypochlorite solution. Hypochlorite feed will be automatically controlled in proportion to overflow rate. Dosage levels required for adequate disinfection would be established by experience, and will probably vary depending on location.

2. Mixing and Artificial Reaeration

The aeration devices were discussed in Part 5. The optimum location of the aerators and the proportion of the total time they will be required to operate will be established by experience. The aerator installations should be arranged to permit easy relocation, and it is expected that their operation can be automatically controlled by appropriately placed dissolved oxygen monitors. Although it is probable that the aerators could be removed from the basin during wintertime, it would be of value to experiment with their wintertime use for de-icing the basin and the enclosed bay. There is a possibility that ice-free conditions could be maintained to provide a wildfowl refuge.

3. Algae Control

Algae growth within the stabilization basin is expected, and would be beneficial in waste stabilization and nutrient removal if properly controlled. It should be emphasized that removal of phosphorous and nitrogen is not a primary objective of the proposed basin, and although any removal of these nutrients, through algae uptake or otherwise will be beneficial, such reduction is considered a secondary benefit.

In consideration of algae growth and control George P. Fitzgerald of the Water Chemistry Laboratory, University of

Wisconsin was consulted. Most of the material of this section is based upon Dr. Fitzgerald's consideration of the proposed project.

Growth of algae in the basin would result in the uptake of phosphorous, nitrogen, carbon and other plant nutrients from the wastewater. In addition, algae growth would result in a rise in pH due to uptake of carbon dioxide, and if the pH rises to a level of about 9.0 or higher, additional removal of phosphorous is possible through precipitation. The algae that are produced in the basin would eventually settle out, and carry the nutrient materials to the bottom along with any phosphorous that has been precipitated. However, the anaerobic conditions expected in the bottom of the sedimentation portion of the basin would result in the decomposition of the algae mass and the other organic sludges, resulting in the re-solution of the phosphorous to the ortho-phosphate form, and the release of nitrogen as ammonia. If the bottom sludges are mixed or agitated, some portion of the soluble nutrients would be returned to the water and could escape in the effluent. Thus, at least part of the anticipated phosphorous removal is illusory unless some method of permanent solids disposal is employed. The question of sludge disposal is considered in the next section of this chapter.

The projected nutrient concentrations in the basin influent indicate that the basin environment will be more nearly that of a stabilization basin than an eutrophic lake. Under these conditions, and assuming the mixing and aeration of the first stage zone, it is expected that the green algae will predominate rather than the blue-greens. Light would probably be the limiting growth factor. In the mixed portion of the basin, with its improved exposure to light, production of algae could probably exceed 400 lbs./acre/week. Very little algae would be expected to settle in the aerobic mixed zone, and this portion of the basin would probably assume a greenish color. Odor or scum problems are generally associated with the blue-green forms, and would not be expected to be a problem in the first-stage zone as long as aeration and mixing is maintained.

In the second stage settling zone, it is expected that nutrient levels would still be ample for algae production. Therefore, it is probable that the upper water layers would remain aerobic, while the lower layers would be anaerobic, at least in the summer months. Blue-green algae might occur in this zone, and if these tend to form objectionable surface scum or odors, chemical control by application of copper or other algicides might be required periodically.

A possible alternative to chemical control would be the aeration-mixing of only the perimeter areas of the settling zone. Most odor problems occur where algae are permitted to cling to exposed surfaces. The vertical walls of the proposed basin tend to minimize the opportunities for build-up, and coupled with perimeter aeration, it is possible that this source of nuisance could be eliminated without use of chemical controls.

Assuming that re-solution of nutrients from the bottom sediments is prevented, it is estimated that under summer conditions, approximately 50 percent removal of phosphorous and nitrogen can be achieved.

Finally, it is anticipated that odor and scum problems, which might tend to occur under some conditions, could be adequately controlled through aeration, mixing, chemical controls, or a combination of these methods.

4. Sludge Disposal

Solid material will be deposited on the bottom of the stabilization basin, particularly in the second stage sedimentation zone. This sludge would be expected to undergo anaerobic digestion, thus reducing the quantity of solids permanently accumulated in the basin. The rate of sludge accumulation is difficult to predict, but preliminary calculations have been prepared on the following assumptions:

(a) Removals of suspended solids in the basin averages 72%.

(b) Net addition of suspended solids by synthesis of dissolved material to cell mass amounts to 10% of the input BOD.

(c) Net addition of solids by algae growth is at an average annual rate of 100 lbs./acre/week.

(d) The weighted average volatile content of all input sludges was calculated at 72%.

(e) Reduction of sludge in digestion is 55%, based on weighted average of the various inputs.

On these assumptions, it is calculated that the net sludge accumulation will amount to approximately 30,000 pounds per day. Assuming a solids content of the sludge deposits of 0.75% for one year, and 1.5% for sludge accumulated for longer than one year, it appears that solids accumulations will amount to about 3.4 percent of the basin volume after 3 years, and 5.2 percent of the basin volume after five years. It is recommended that sludge be removed for disposal at intervals of 5 to 10 years, if these preliminary projections are verified in operation.

Disposal of the accumulated sludge may be done in various ways. The most economical and satisfactory alternative is to pick up the material by hydraulic dredge and discharge it into the permanent solids retention impoundment tentatively proposed by the U.S. Corps of Engineers for disposal of material dredged from the Cuyahoga River and Cleveland Harbor. At the present time, the Corps of Engineers is carrying on a study of the permanent impoundment of dredging spoil at various Great Lakes ports. Part of this proposal involves the construction of an offshore impoundment at Cleveland, similar to the proposed stabilization basin, and located at approximately the same site. The Corps impoundment would be designed to retain the solids from dredgings, and to decant the supernatant liquid to some suitable site for treatment prior to discharge to Lake Erie. It is obvious that these two projects could be mutually beneficial. The stabilization basin, being designed for treatment of wastewater, could receive the supernatant liquid decanted from the Corps' impoundment. In turn, the Corps impoundment, being intended for solids retention, could receive for permanent disposal the solids dredged from the stabilization basin.

Not only would this arrangement be mutually beneficial in operation, but it also presents a possibility for substantial savings in construction cost. The Corps is contemplating construction of a basin similar to that proposed herein for the stabilization basin. If the two projects could be coordinated in the design stages, they could be planned to be adjacent to one another, and constructed with a common dividing wall. It is estimated that the savings in cost using common wall construction would approximate $10,000,000. In addition to this savings, an annual reduction in operating and maintenance costs would be realized through the mutual exchange of waste materials for disposal. These possibilities have been discussed with representatives of the District office of the U.S. Corps of Engineers, and have received favorable preliminary consideration; a final determination on its feasibility must await completion of the Corps' studies on disposal of dredging spoil.

One alternative to the disposal of sludge in the Corps' impoundment is to pump sludge from the stabilization basin to the Easterly Plant for transmission through the existing sludge force main to the Southerly Treatment Plant for dewatering and incineration. At the present time all sludge wasted at the Easterly Plant is disposed of in this manner.

A second alternative is permanent retention of sludge within the basin. It is probable that a part of the 45 foot depth of the basin could be devoted to permanent sludge storage, provided that the sludge could be adequately sealed off from contact with the overlying water to prevent release of nutrients. Experimental work has been conducted in smaller oxidation basins on the sealing of the sludge-water interface by use of a thin layer of sand or earth fill, or by use of light vinyl membrane sheets. Preliminary results appear encouraging. Such membranes might provide effective seals, and still allow consolidation of the deposited sludge.

The final choice of the most effective and economical method of sludge disposal cannot be made until the dredgings impoundment project has been evaluated by the Corps of Engineers.

5. Scum Removal

It is anticipated that floating debris and algae may occasionally tend to form a surface scum, particularly in the second stage settling zone of the basin. Since the basin effluent will be withdrawn for discharge from about 10 feet below the surface, it is expected that scum will not be discharged with the effluent, and will not create a problem in Lake Erie.

However, scum accumulations may be the source of odors, and are certainly unsightly within the basin. Since this should be a problem of minor proportions, automatic scum removal equipment is not economical. To prevent a nuisance condition from this source, regular boat patrols should be carried out for removal of scum and floating debris.

6. Surveillance, Algae Control, Maintenance

The same work boat used for scum removal would be available for routine control and maintenance procedures. These would include regular inspection and maintenance of the basin aerators, sampling for laboratory control, maintenance and adjustment of inlet and outlet ports, and occasional spreading of algicides for control of algae if necessary. Laboratory services for basin control would be available at the Easterly Plant, and it is not believed necessary to provide additional facilities.

Maintenance of the basin enclosure wall is expected to be nominal. The principal maintenance factor is expected to be repair of the cofferdam cap, which is subject to weathering and physical damage by wave action. Very little mechanical equipment is involved in the basin itself, so that maintenance is expected to be a minor item.

PART 8

SECONDARY BENEFITS AND OTHER CONSIDERATIONS

The preceding parts of the report have described the principal objectives of the proposed stabilization basin, and the anticipated benefits related to water pollution abatement. In this section some of the possible secondary benefits and other factors involved will be considered.

1. Erosion Protection

At Cleveland, the shoreline of Lake Erie generally takes the form of a bluff varying from 10 to 50 feet in height, with a narrow strip of beach at its foot. This shoreline bluff has been undergoing serious erosion for many years, and in some places, formerly valuable lakefront property has been almost completely destroyed. The extent of this bluff recession, and various proposals for control of the problem have been extensively studied(23)(24). In a report published in 1950, the U.S. Corps of Engineers(25) states that in the eastern portion of Cleveland adjacent to the proposed basin, the landward movement of the shoreline has been 130 to 150 feet in the period 1876 to 1947; much more severe recession has taken place since 1947 due to the high lake levels of the 1950's. In their study, the Corps of Engineers prepared preliminary plans and estimates of cost for shore protection and erosion control facilities. The estimate of cost in 1947 was $5,055,000. At 1968 construction cost levels, this shore protection program for the easterly part of Cleveland would cost approximately $12,500,000. It was further estimated by the Corps that the annual total benefits in terms of damages prevented and additional recreational benefits would amount to $371,000, which would be equivalent to $552,000 per year in 1968 dollars.

The proposed stabilization basin would thus replace facilities for beach and erosion protection valued at about $12,500,000 in capital cost plus $500,000 annual benefit to the shoreline properties, both public and private.

2. Recreational Benefits

The proposed configuration of the stabilization basin would create a sheltered bay in Lake Erie, between the inner wall of the basin and the beach, about a mile and a quarter long (along shore) and extending one-half mile out into the Lake. This bay would provide a prime recreational asset in Cleveland, for all kinds of water-based recreational activities. At the eastern end of this bay lies the present White City Beach, which is a small public bathing beach. This beach could be extended westward within the basin to whatever extent may be desirable for public recreational uses.* At the present time, bathing beach facilities are entirely inadequate in Cleveland, and expanded clean beaches are sorely needed. The bay would be ideally suited for this purpose, since it is conveniently located and within easy access of the central city. No sewer outlets or streams would enter the bay, and water quality control could be carried out if required to maintain excellent bathing water quality.

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*The Bureau of Outdoor Recreation, U.S. Department of Interior, has certain federal funds available for acquisition of new land for public beach and recreational purposes.

Ample space is available within the bay for boating, fishing and water skiing, without interference with the bathing beach areas. A marina could be ideally situated within the bay, built contiguous with the basin wall. The cellular basin wall is wide enough to carry a two-lane roadway, and parking areas, picnic areas and fishing facilities could be incorporated at reasonable cost. In order to provide an aesthetically pleasing aspect from the shore, it is proposed to plant trees and grass in the tree-lawn area of the roadway, and between the parking areas. Although some annual replacement of plant materials would probably be required, this may be justified in terms of aesthetic acceptance of the entire project.

An artist's conception of the stabilization basin illustrating the bay area and recreational facilities is shown in the frontispiece.

3. Nutrient Removal and Chlorination

The proposed basin contains ample space for chlorine contact for disinfection of both plant effluent and storm overflows. At the present time, the Easterly Treatment Plant has no facilities for chlorination of bypassed and storm overflow waters. As part of a program of improvements and enlargements now being carried out at the Easterly Plant, a chlorine contact chamber has been proposed in the project plan(26) recommended by the City's consulting engineers. The preliminary estimate of cost of this chlorination facility was $3,000,000, expenditure of which would be unnecessary if the basin were constructed, thus saving this amount on the Easterly project.

In addition, there exists a possibility that high degree nutrient removal processes might be required at the Easterly Plant within the near future. Such facilities would require substantial land area, and either private property adjacent to the plant site would have to be purchased, or the plant would have to be extended by reclaiming land within the Lake. The area enclosed by the basin would furnish a suitable site for such nutrient removal facilities, at much lower cost.

4. Wildlife

The proposed basin and the bay created by the basin would afford an opportunity for various benefits for wildlife activities. Representatives of the Ohio Department of Natural Resources have indicated an interest in the project from the standpoint of a possible winter wildfowl refuge. Other related activities are possible.

The impoundment itself would be large enough to afford an opportunity for various activities and experimental programs in aquatic biology, algae control, and similar studies.

5. Possible Related Projects

There are two proposed projects which could have a bearing on the proposed basin. As discussed in Part 7, the U.S. Corps of Engineers is presently studying the feasibility of a similar offshore impoundment for retention of dredging spoil. It was pointed out that, if the Corps' project were constructed, the two projects would be mutually beneficial and complementary, and considerable cost savings could be realized.

A second related project is the proposal by the City of Cleveland in 1966 that a major limited access freeway be constructed on fill in Lake Erie, offshore of Cleveland. The proposed highway would traverse the general area of the stabilization basin, and if the proposal were adopted, the highway and basin projects could be advantageously coordinated. The main advantage would be the reduction in construction cost which might be realized by use of common wall construction, and elimination of duplication of the breakwall. This possibility should be pursued in future consideration of both projects.

6. Legal Aspects

Although the scope of work of this contract does not include detailed examination of the legal implications and procedures of the proposed basin, preliminary consideration has been given to those implications and procedures.

The basin would be constructed entirely within Lake Erie, and in United States waters. Title to the land within the lake beyond low water datum elevation lies in the State of Ohio. The approval of the Ohio Department of Natural Resources would be required for installation of the basin, together with issuance of a permit for construction. That agency is also responsible for consideration and approval of the project from the standpoint of Lake Erie as a water resource, and the possible effects related to wildlife and recreation.

The pollution aspects of the project would require the approval and permit of the Ohio Department of Health, through the Ohio Water Pollution Control Board.

We have held preliminary discussions of the project with representatives of the Ohio Department of Natural Resources, and the Ohio Department of Health. At the preliminary stage of discussion, it is of course not possible to obtain a firm commitment from these agencies, and none was sought. However, the reaction to the proposal from both departments was quite favorable, and no obstacles, either legal or technical, were discovered which would prevent the development and construction of the project.

The U.S. Corps of Engineers, Buffalo District, is also officially involved with construction in Lake Erie. The Corps is required to review all such projects constructed in navigable waters, and to make a determination on the desirability of such a project, taking into account its effect on navigation, adjacent properties, and other aspects and uses of the waterway. This determination normally involves an investigation and public hearing held by the Corps. If the project receives approval of the Corps, a permit for construction is issued. We have also held preliminary discussions with the Corps' District

Engineer in Buffalo, and with members of his engineering staff concerning this project. These discussions revealed no specific obstacles or objections known at this time, and the general reception of the project was positive. Again, no approval or affirmative commitment is appropriate prior to official application to the Corps, and its review.

With respect to the shoreline collection system, certain of the contemplated facilities will require approval by various departments of the City of Cleveland, and certain of the facilities may require purchase of private property or easements from private owners. In general, it has been found most economical to construct the collector conduits in the shallow water of the Lake rather than on the higher bluff - and this will minimize the interference and encroachment on private property. The extent of such easements or purchases cannot be determined until detailed design is undertaken, but this is not expected to be a major problem. An allowance has been included in the estimates of cost to cover easements and rights-of-way.

PART 9

ESTIMATES OF COST AND CONSIDERATION OF FEASIBILITY

1. Estimated Costs

Preliminary estimates of construction cost have been prepared for the proposed stabilization basin and for the shoreline collection system. The estimates were based on 1968 construction cost levels in Cleveland, and were made on computed quantities of work based on the preliminary design discussed in Parts 5 and 6. It should be noted that the estimates are subject to variations because actual subsurface conditions are not yet known.

Because of the unusually large scope of the project and the nature of the work, it was deemed advisable to discuss the construction of the basin with general contracting firms experienced in large marine construction projects. Preliminary sketch plans and specifications were prepared and submitted for their review, and various aspects of the construction methods and equipment were discussed. Following these discussions, preliminary cost estimates made by the contractors were received, and found to compare favorably with our independent estimates.

These preliminary construction cost estimates include all of the facilities, mechanical and electrical power, gate lock, causeway, cap and roadway, in addition to allowance for mobilization, contract bond, and contractor's overhead and profit, and are based on 1968 price levels, referenced to the Engineering News Record Construction Cost Index for Cleveland of 1440.

In the preliminary estimates and amortization calculations, we have made no allowance for participation in financing by any agency of the Federal government or the State of Ohio. It is considered likely that if the project is adopted as feasible by the City and these agencies, some financial participation outside of Cleveland will be required in order to successfully fund the project. The City of Cleveland has a Master Plan program of pollution abatement involving over $50,000,000 in capital improvements, exclusive of the offshore basin project, and the financial resources available to the City are probably not sufficient to undertake the entire program without federal or state aid. For purposes of this report, however, the effect of possible aid programs is not considered.

To obtain estimates of total project cost, an allowance of approximately 15 percent was added to the construction cost. This allowance includes contingencies, cost of easements and right-of-way, permits, engineering, testing of materials, supervision of construction, legal and administrative costs and miscellaneous overhead charges.

The summary of the total project costs thus computed is given in Table 9-1.

TABLE 9-1

ESTIMATED CONSTRUCTION AND PROJECT COSTS

Item	Estimated Total Construction Cost	Estimated Total Project Cost*
1. Offshore Stabilization Basin, including causeway, roadway and other appurtenances	$52,600,000	$60,490,000
2. West Branch Collector System, including collector basins, pumping stations and submerged outfalls to offshore basin	11,700,000	13,455,000
3. East Branch Collector System, including collector basins, pumping stations, and submerged outfalls to offshore basin	8,300,000	9,545,000
TOTAL COST	$72,600,000	$83,490,000

*Construction cost plus 15%±.

2. Operation and Maintenance Costs

Annual operation and maintenance costs have been estimated for the project. These costs include all normal elements of operation, and are based on current wage and cost levels prevalent in the Cleveland area. The estimates include costs of operation and maintenance of the basin and the West and East branch collector systems, and are shown in Table 9-2. The item of labor and supervision includes the necessary disposal of screenings, pumping station operation, basin operation, and laboratory control. Materials and supplies include cost of hypochlorite solution for disinfection and an allowance for algicides. Power for pumping and aeration-mixing is included, and maintenance charges contain an allowance for annual repair of the capping material of the wall.

TABLE 9-2

ESTIMATED OPERATION AND MAINTENANCE COSTS

Item	Estimated Annual Cost
1. Labor and Supervision	$ 89,000
2. Materials and Supplies	28,000
3. Power and Fuel	51,000
4. Maintenance	62,000
TOTAL ANNUAL O & M COST	$230,000

3. Total Annual Cost

Total annual cost is the sum of the cost of amortization of capital expenditure and annual operation and maintenance cost. In considering amortization, it is recognized that the useful life of this project will greatly exceed the normal 20-25 year life often assumed for treatment plant facilities, which involve substantial investments in mechanical equipment. It is likely that the main elements of the basin and collector systems will have a useful life exceeding 50 years. A capital recovery factor for calculation of amortization costs has been selected at 40 years, with interest rate of 4.5 percent.

The total annual costs are shown in Table 9-3.

TABLE 9-3

TOTAL ANNUAL COST

	Item	Capital Project Cost	Annual Amortization Cost*	Annual Operation and Maintenance Costs	Total Annual Cost
1.	Offshore Stabilization Basin	$60,490,000	$3,287,000	$111,000	$3,398,000
2.	West Branch Collector	13,455,000	731,000	56,000	787,000
3.	East Branch Collector	9,545,000	519,000	63,000	582,000
	TOTAL	$83,490,000	$4,537,000	$230,000	$4,767,000

*Capital recovery factor 0.05434 based on 40-year, 4-1/2%.

4. Consideration of Feasibility

The determination of feasibility of this project involves consideration of many factors, some of which are not amenable to dollar evaluation. The primary purpose of the project is abatement of the pollution of Lake Erie, an objective which is not directly measurable in money, since many of the elements of pollution abatement involve health, esthetic enjoyment, preservation of various imponderable values for future generations, and subtle effects of water quality for many uses, none of which can be directly evaluated. For this reason, calculation of a cost-benefit ratio is not an appropriate means of viewing feasibility in projects designed for control of pollution. Similarly, the present project would result in additional benefits for recreational enjoyment, which can be only approximately evaluated.

The feasibility of this type of project, then, rests upon the determination of its effectiveness in accomplishing its objectives, and on the comparative costs of satisfying these objectives by other alternative methods, if such are available.

During progress of the studies carried out for the City of Cleveland in preparation of the Master Plan for Pollution Abatement, a detailed examination was made of the cost of sewer separation in Cleveland. Four large subdistricts of the Cleveland system were analyzed in detail for separation of storm and sanitary sewage flows, preliminary designs of separate sewers were prepared, and detailed cost estimates were made. The average cost of separation in Cleveland was estimated to be $21,400 per acre, for a total cost within the City of Cleveland of $948,000,000.

The proposed stabilization basin would serve about 13,000 acres of the Easterly combined sewer district of the Cleveland system. At the estimated cost of $21,400 per acre, cost of sewer separation in this service area would total $278,000,000, or about 3 times the cost of the proposed basin. However, this is not a complete comparison, since the basin would actually do considerably more to abate pollution than would complete separation of sewers, by collecting and treating all of the surface drainage from the service area. The data given in Part 2 indicates the heavy load of polluting materials discharged by these streams into Lake Erie, about one-third of which originates in the suburban systems which would not be improved by the separation of Cleveland's combined sewers. It is clear that the proposed stabilization basin would actually accomplish a considerably higher degree of pollution control than sewer separation and at about thirty percent of the cost. It may be concluded that the basin is definitely an economical and feasible alternative to sewer separation in Cleveland.

Viewed as a treatment process, the stabilization basin is a feasible project. The probable removals of BOD, solids, bacteria and nutrients discussed in Part 5 represent an effective tertiary treatment of the Easterly Plant effluent as well as combined sewer overflows. Based on actual treatment efficiency at the Easterly Plant and the projected removals in the basin, the overall removal of BOD and suspended solids in the raw sewage received at Easterly (including removal in the basin) would be 96 percent and 97 percent, respectively. This compares with removals in the present plant in recent years of 89 percent BOD and 85 percent suspended solids. With respect to nutrients, overall removal of phosphate would be improved from the present 42 percent to about 70 percent, while removal of nitrogen would be improved from 18 percent to approximately 58 percent. The high expected removal of coliform and streptococci organisms has been discussed previously.

These projections indicate an effective tertiary treatment process, and an even more effective overall process for treatment of the combined sewer overflows. A review of the literature regarding tertiary treatment indicates that the total cost of plant treatment processes capable of providing equivalent results would fall in the range of $35.00 to $45.00/mg for the Easterly effluent, and $45.00 to $50.00/mg for overall treatment of the combined sewer overflow. Using an average of $45.00/mg for the total flow, the total annual cost of treatment by new plant devices would be about $4,500,000 per year. In addition, cost of purchasing additional land or reclaiming land from the lake would have to be added, which is estimated at about $5,000,000, or $270,000 per year if amortized over 40 years. An alternate process would require the same shoreline collector system to convey sewer overflows and stream flow to the treatment site. A comparison of the alternate costs is shown in Table 9-4.

TABLE 9-4

COMPARATIVE TOTAL ANNUAL COSTS OF TREATMENT

Item	Stabilization Basin*	Alternate Plant Process @ Easterly
East and West Collectors	$1,369,000	$1,369,000
Treatment facilities	3,398,000	4,500,000
Land cost	---	270,000
TOTAL	$4,767,000	$6,139,000

*From Table 9-3.

Admittedly, the figures shown for an alternate process at Easterly are an approximation only, since detailed studies of an alternate was not included in the scope of work of this contract. However, the approximation of Table 9-4 indicates that there is a large cost differential in favor of the stabilization basin, probably in excess of $1,000,000 annually.

Not included in this comparison are the secondary benefits of the basin, and as indicated in Part 8, the secondary benefits are substantial. The Corps of Engineers' estimate of the cost of shore protection projects which would be replaced by the basin amounts to about $12,500,000 in capital cost (1968 dollars) and the additional annual benefits to public and private shoreline property was estimated at $500,000. If these benefits were credited to the basin, the total annual cost of the basin would be reduced by $1,180,000, and the cost of alternative treatment facilities would exceed the cost of the basin by more than $2,500,000 annually.

The proposed stabilization basin provides other tangible and intangible benefits. At the present time, the Easterly Plant effluent is discharged directly at the breakwall line, and effluent dispersion and dilution are inadequate. The effects on coliform counts at nearby White City beach have demonstrated this fact. The Cleveland Master Plan program includes a recommendation that if the stabilization basin were not built, a submerged outfall sewer would be required to achieve adequate compliance with the Lake Erie water quality standards with respect to coliform concentrations, localized depletion of dissolved oxygen, and solids discharges. Total project cost of the outfall, including pump station, was estimated at $5,500,000, another charge which can be credited to the stabilization basin as a savings.

The stabilization basin has been found feasible from the standpoint of structural design and construction. Although a project of this magnitude requires detailed design studies concerning alternative foundation designs and methods of construction, the preliminary design indicates that there is no feature of the site or element of structural stability that would render the project infeasible.

With respect to basin operation and control, the project is also feasible. There are no operating problems anticipated of significance, and basin management is expected to be relatively simple, particularly when compared to more complex chemical and physical processes available for tertiary treatments. Adequate surveillance and control of algae and floating materials appear to be the main elements required to prevent nuisances, and such procedures are not expected to involve difficult or extraordinary efforts.

The project is feasible from the standpoint of lake navigation and its effects on the adjacent property. Preliminary discussions indicate that the designs shown in Figure 5-1 will not create a significant navigation problem, and if minor problems should arise during public discussion of the project, the shape of the basin can be modified within reasonable limits to overcome them. Its effect on the shoreline properties is largely beneficial, since the basin will provide a ready-made breakwall and protected zone which will largely eliminate the costly erosion of the shoreline. In addition, the creation of a sheltered bay provides an extremely valuable recreational asset which would be of direct benefit to shoreline property owners, both public and private.

In summary, considerations of all aspects of the proposed basin indicate that the project is feasible, and offers a number of benefits at lower cost than other available alternatives.

5. Possible Reductions in Scope of the Project

The Statement of Work of our contract for this study includes under paragraph 9 the consideration of an alternative stabilization basin project of smaller size to serve a lesser area. The purpose of this provision was to obtain cost estimates of a project of smaller scope which may be more nearly applicable to other locations, and which would possibly provide a feasible first stage project if the entire project were to be divided into two or more stages.

During progress of the work, it was agreed with representatives of the Federal Water Pollution Control Administration that this alternate basin would be designed on the following basis:

(a) The smaller basin would be designed to receive only the flows from the Easterly treatment plant and the Easterly combined sewer overflow.

(b) Design criteria for the plant effluent flow would be based upon the present average flow (125 mgd) rather than the ultimate (151 mgd) flow.

(c) Storm overflow rates would be limited to the hydraulic capacity of the existing overflow and by-pass conduit at the Easterly Plant.

(d) Design criteria as to detention period, inlet and outlet features, etc., would be the same as those for the main basin, except that allowance for future increase in runoff rate and for wave action would not be included.

On the basis, the capacity of the alternative basin was calculated at 14,000 acre-feet, (about 47 percent of the volume of the main basin). Area would be 410 acres at a mean depth of 34 feet.

The smaller basin should be designed to accommodate expansion to its ultimate size in the future. For that reason, the basic shape of the east portion near the Easterly plant was retained, and a westerly wall was added. The perimeter length of the modified basin was 19,000 lineal feet, almost two-thirds that of the main basin.

Estimates of cost were prepared on a basis similar to the original costs. Cost of the shoreline collector system is, of course, not included. The total construction cost of the alternative was calculated at $40,000,000 and the total project cost at $46,000,000, which is 76 percent of the cost of the full scale basin.

If the full project were built eventually, the west wall of the smaller basin would be duplicated, and the overall project cost would be increased by about $12,000,000, not including additional costs due to escalation during the intervening period.

The smaller alternative basin has a basic undesirable feature in addition to higher overall cost, which is the effect on the Cleveland shoreline. The smaller basin would not eliminate the combined sewer overflows and polluted streams which now are the major sources of beach pollution. These discharges would continue to occur directly at the beach line, and although an improvement in the total pollution picture at Cleveland would be made, the most immediate and pressing problem would not be solved.

For these reasons, we believe that the alternative smaller basin should not be undertaken unless every effort at carrying out the main basin had failed.

In conducting the work of this alternative basin, consideration was given to one other modification in the overall project. The estimates of cost presented in Tables 9-1 and 9-3 show one element of the overall full scale project that appears to involve costs which are high when compared with the water quality improvements likely to be realized. This element is the East branch collector system. Study of the input load data presented in Part 2 shows that the loads from the East branch system amount to only about 2 percent of the input BOD and 14 percent of the input suspended solids. Part of the suspended solids loads to Euclid Creek originates in the discharge of filter plant backwash water which will be eliminated through other programs. At the present time, Euclid Creek, which is the major flow into the East collector system, is reasonably clean stream. Much of the tributary area of Euclid Creek is undeveloped land, and a substantial part of that which is developed has separate sewers which reduce the amount of polluting materials discharged.

The cost of the East branch collector system represents about 12 percent of the total project cost, compared with the approximately 5 percent of the pollution load discharged. Furthermore, discharges from the East branch district are normally dispersed into the offshore waters more readily than those of the West system, because of the configuration of the shoreline, and their effect upon the beaches is less because of the normal eastward set of the lake currents. For these reasons, it appears reasonable to defer construction of the East branch collector system. If further development of the Euclid Creek system results in heavier pollution loads being discharged, the East branch collector could be constructed when deemed necessary. The stabilization basin should be designed and constructed to include volume allocated to the East branch system, since later additions to the basin would be costly in relation to the volume added.

ACKNOWLEDGEMENTS

This feasibility study was carried out by Havens and Emerson under Contract No. 14-12-27 for the Federal Water Pollution Control Administration, Department of the Interior.

The stabilization basin project was originally conceived by Mr. John J. Wirts, Superintendent of the Easterly Pollution Control Center, whose advice and assistance throughout the project is gratefully acknowledged.

The project has benefitted by the cooperation of the City of Cleveland, through the offices of Mr. Walter E. Gerdel, Commissioner of Water Pollution Control, and Mr. Raymond Roth, Superintendent, Bureau of Industrial Wastes.

Dr. George P. Fitzgerald of the University of Wisconsin served as consultant to the engineers on matters concerning algae control and biology of the basin. We were fortunate to obtain the assistance of this eminent scientist.

In the office of Havens and Emerson, the project was supervised by Lamont W. Curtis, Project Engineer, under the direction of George D. Simpson, Partner, Project Director.

APPENDICES

APPENDIX A

HYDROLOGY

Additional details of the collection and analysis of the rainfall and runoff data are presented herein. Discussion of the rainfall data is followed by discussion of the runoff date collected. The use of these data in the development of design hydrographs is also presented.

RAINFALL

In this section, the collection and evaluation of rainfall data and development of design rainfall depths is discussed in the following paragraphs.

Collection of Rainfall Data

Background data for the Cleveland area precipitation were available from the U.S. Weather Bureau at Cleveland Hopkins Airport. Additional gauges were installed at two locations within the drainage area to provide rainfall data for comparison with runoff data in constructing unitgraphs and for computing peak rates of runoff in the design storm. As part of a study for the City of Cleveland, Havens and Emerson had installed three additional gauges. The City of Cleveland Heights had two recording rain gauges and cooperated with Havens and Emerson and the City of Cleveland by allowing use of this collected data. The U.S. Weather Bureau has two established stations, one at the previously mentioned Cleveland Hopkins Airport and a second one at the Easterly Wastewater Treatment Plant. The total rain gauge network provided excellent rainfall data for constructing unitgraphs, investigating depth-area distribution, and intensity-area distribution.

The recording rain gauges were manufactured by Belfort Instrument Company and were the official U.S. Weather Bureau weighing and continuous recording type. Strip charts were changed daily and were geared for one complete revolution of the chart in a 24 hour period. Time could be measured to 5-minute intervals and was checked daily by the individual who changed the chart. The recording pin was set to record 1 inch of rainfall depth as 1 inch of pen movement on the chart.

Charts were analyzed, and hourly data were taken off to conform to the methods of the U.S. Weather Bureau. Excessive precipitation was also recorded for durations of 5, 10, 15, 20, 30, 60 and 120 minutes. The charts were used directly in obtaining rainfall intensity and durations in construction unit-graphs and computing percentage of runoff data.

Frequency - Precipitation Data

The long term data from the Cleveland Hopkins Airport was used to determine frequency precipitation data. Frequency-depth-duration curves and frequency-intensity-duration curves were made from excessive precipitation data of 76 years of record for 5, 10 and 60 minute duration, and 63 years of record for 15, 30 and 120 minute duration. Excessive precipitation is defined as rainfall exceeding a depth for a definite duration and the depth is based on the formula 0.20 inch + (.01 inch) (T) where T is the duration. The following table results:

TABLE A-1

EXCESSIVE PRECIPITATION

Duration (minutes)	Depth (inches)	Intensity ins./hr.
5	0.25	3.0
10	0.30	1.8
15	0.35	1.4
30	0.50	1.0
60	0.80	0.8
120	1.40	0.7

A frequency-duration-depth array was developed for all of the excessive precipitation by listing depths in a descending order of magnitude; frequencies were computed; and the data plotted. Figure A-1 is the result of this plotting for 1 and 2 hour durations.

Figure A-2 is the same data except that intensity has been plotted against duration for various frequencies. In order to extend the curves into the range of low intensities, that is, rainfall which is not excessive, an array of frequency-duration-depth was developed for hourly rainfall data. The data used were from the hourly records as published in "Local Climitalogical Data Bulletins - Cleveland, Ohio, U.S.W.B.". All rainfall greater than a trace was incorporated into this array. Since these data were in hourly form, the durations studied had to be limited to hour units. Durations investigated were for 1, 2, 3, 4, 6, 8, 12, 18 and 24 hours of mutually exclusive storm events, for the period of June 1950 to December 1966. Since much of the winter precipitation is in the form of snowfall, the data were arranged in two ways, one array for the months of March through October and the second array for all twelve months. Figure A-3 shows the results of plotting the twelve-month array.

These data are required in constructing standard design storms and computing peak rates of runoff. In addition to this, data was needed on the frequency of depths for duration longer than 24 hours and not necessarily for storms of continuous precipitation.

Frequency of Depths - Daily or Longer

The daily precipitation data for the period of 1950-1966 was put in an array to determine frequency of total daily depths. In addition, the daily precipitation data were grouped in periods of 2 days, 3 days and 30 days and arranged in arrays to determine frequencies and depths of rainfall for 1, 2, 3 and 30 days. The monthly or 30-day data are more fully described in the next section. From this array of daily or longer total depths, the following table results:

TABLE A-2

FREQUENCY - DEPTH OF DAILY OR LONGER PRECIPITATION

Depth of Rainfall - Inches

	Duration				
Frequency	2 hours*	1 day	2 days	3 days	30 days
1 - Year	1.0	1.8	2.0	2.2	4.8
2 - Year	1.3	2.2	2.4	2.6	5.7
3 - Year	1.5	2.4	2.6	2.8	5.8
5 - Year	1.7	2.7	2.9	3.1	6.3
10 - Year	2.0	3.1	3.3	3.5	6.6

*Continuous Rainfall

Monthly Precipitation

The monthly precipitation rates for the point rainfall station at Cleveland Hopkins Airport were investigated for average, median, range and standard deviation for the period of 1931-1966. The data was investigated for all months of the year and for the month of March through October for the same period. Table A-3 shows the results of this investigation. As shown on the table, the monthly average with all months considered as 2.8 inches and with only March through October considered, the average rises to 3.0 inches. This data will be used further in the discussion to determine the amount of storage to be allocated to runoff from rainfall over the area tributary to the stabilization basins.

TABLE A-3

MONTHLY PRECIPITATION VALUES - INCHES

Month	1871-1966 Average	S^{*1}	1931-1966 Average	Median	Range	
					Min.	Max.
January	2.57	.85	2.54	1.96	.36(*2)	7.01
February	2.37	.41	2.27	2.19	.73	4.64
March	2.88	.54	3.07	2.89	.78	6.07
April	2.79	.56	3.30	3.28	1.13	6.61
May	3.07	.52	3.10	2.94	.58	6.04
June	3.22	.61	3.14	3.02	.39	6.64
July	3.38	.36	3.09	3.26	.96	5.37
August	2.96	1.04	3.28	3.20	.72	9.38
September	3.06	.64	2.73	2.58	.74	6.37
October	2.61	1.30	2.32	1.78	.61	9.50(*3)
November	2.58	.70	2.55	2.30	.92	6.44
December	2.36	.54	2.32	2.41	.71	5.60

*1 - Standard Deviation 1931-1966 period
*2 - Minimum Monthly 0.36-Oct.
*3 - Maximum Monthly 9.50-Jan.

Median of Maximum Monthly	6.22"/month	
Median of Minimum Monthly	0.96"/month	
Maximum Daily	3.73" (May)	
Median of Maximum Daily	2.53"	
Monthly Average Period	1871-1966	2.82
	1931-1966	2.81
Monthly Average Excluding Jan., Feb., Nov., and Dec.	1871-1966	3.00"
	1931-1966	3.00"

Design Storm for Peak Rates of Runoff

The collected precipitation data were valuable for preparation of the unitgraph and to determine runoff characteristics of the subdistrict drainage area. After the unitgraphs are constructed and verified, they must be projected to give peak rates of runoff from the drainage area for the frequency of design used to determine the hydraulic capacity of the collection system. Both peak rates of flow and volume of runoff under the design hydrographs are needed because pumping is involved, and storm flow routing techniques were used to optimize the collection system.

The design storm of the collection system would, of economic necessity, be low. Consequently, design storms for the Cleveland area were constructed for 1, 3, 5 and 10 year rainfall frequencies. In order to construct a design storm hyetograph, the length of storms and period of maximum intensity in storms had to be investigated.

Length of Storms

Mutually exclusive storms were listed by durations to give an indication of the length of storms. This was done for all months; March through October; and winter months; (November, December, January, February), for a 16-year period of record to determine a base length for a design storm. All storms for the 16 year period were listed and put in an array by order of magnitudes for the various durations. The results of this are plotted on Figure A-4, and 6 hours was chosen as the design storm length.

Period of Maximum Intensity

About 30 storms of various intensities were closely examined to determine the locations of maximum intensity within the total duration for determining the shape of a design storm. The storms were of various durations ranging from 48 minutes to 26 hours and were selected at random. Each storm was divided into equal durations, and the percentage of rainfall depth of the total rainfall was found for the respective percent of the total duration. All depths and durations were subsequently compared on a percentage basis.(27) The data were plotted as a mass diagram for each storm. The slope of the curve then indicated rate of rainfall. An average curve was developed from all of the storms by adding the values of depths at every 10 percent value of time. The average curve had a maximum slope occurring at 38 percent of the total duration. This indicated that the maximum intensity is occurring at 38 percent of the time distribution of the storms. This compares with 37.5 percent found in Chicago(28) and 45 percent found in New York.(29) The results are shown in Figure A-5.

Design Storm Hyetographs

The design storm mass diagrams were developed by plotting accumulated depths of rainfall versus time. The slope of the resulting curve is the intensity. The depths of rainfall were taken directly from the frequency-depth-duration curves. The total depth of rainfall for all durations up to six hours for a selected frequency were plotted around a centerline, so that the depth of rainfall for 1 hour of the select frequency occurred between the 2-1/2 to 3-1/2 hour period of the 6 hour rainfall and the mid-depth of the 1 hour rainfall occurring at the same mid-depth of the 6 hour rainfall. Each duration was likewise plotted so that the resulting curve was a mass diagram of a rainfall with a 1, 3, 5 or 10 year frequency. The maximum duration of 6 hours was selected as being the design storm length as previously described.

The hyetographs and mass diagrams thus developed were adjusted so that the maximum intensity occurred at 38 percent of the time duration, to conform with the pattern of the observed storms. These adjusted mass diagrams for 6 month, 1-year, 3-year, 5-year and 10-year storms, are shown on Figures A-6. The maximum depths and intensities for various durations during the design storm are given in Table A-4.

These developed design storms were then used with the unitgraphs to determine the peak rates of flow for the collection system and volumes of runoff for design of the pumping stations and holding basins.

Areal Distribution

The recording rain gauges located throughout the drainage area provided valuable information in determining the extent of areal distribution. Marston(30) and Linsley and Kohler(31) have shown that areal distributions can be significant. Variations between excessive rainfall measured at a point source and compared to the other point source rainfall values have been compared herein for the different areas of rainfall patterns. In general, the findings show agreement with Marston's findings in the Boston area.

Marston's and Havens and Emerson's data are compared in Table A-5 with the addition of a ratio for 8,000 acres.

TABLE A-4

DESIGN STORM

Frequency	15 Min.	30 Min.	1 Hr.	2 Hr.	4 Hr.	6 Hr.
	Maximum Depths for Various Durations (inches)					
6 Months	.47	.56	.66	.82	.86	.90
1 Year	.60	.78	.90	1.04	1.08	1.14
3 Years	.86	1.10	1.30	1.46	1.50	1.58
5 Years	.99	1.28	1.50	1.66	1.70	1.80
10 Years	1.13	1.55	1.80	2.10	2.20	2.30
	Maximum Intensities for Various Durations (in./hr.)					
6 Months	1.87	1.12	.66	.42	.22	.15
1 Year	2.42	1.56	.90	.52	.27	.19
3 Years	3.42	2.20	1.30	.73	.37	.26
5 Years	3.96	2.56	1.50	.83	.42	.30
10 Years	4.52	3.10	1.80	1.05	.55	.38

TABLE A-5

RATIO OF OVERALL AREA RATE TO MAXIMUM POINT RAINFALL

Area	30 Min. Duration		60 Min. Duration		6 Hr. Duration
	Marston	H&E	Marston	H&E	H&E
Pt. Rainfall	1.0		1.0		1.0
1,000 Acres	.90		.95		
2,000 Acres	.85	.88	.93	.90	.97
4,000 Acres	.80	.78	.88		.96
8,000 Acres	.75		.85		.93
10,000 Acres					.92
20,000 Acres					.87

Table A-5 resulted from drawing isohyets (lines of equal rainfall) for the observed depths at the stations over the study area. The average depth of rainfall was determined by computing the incremental volume between each pair of isohyets, summing these volumes and dividing by the total area. This average rainfall was determined for storms of various durations and the ratio between the overall rate and the maximum point rainfall was determined. This ratio was applied to adjust the peak discharge computed by the rational method Q=KCIA where K is the aforementioned ratio.

DESIGN STORM FOR POLLUTION LOADS

Not only did a design storm have to be developed for determining the peak rates of runoff for the design of the collection system, it was also necessary to develop a design storm which could be used for assessing pollution load discharged to the basin from an excessive storm occurring during a 30-day average design period. The average monthly precipitation for the year is 2.8 inches of rainfall (3.0 inches for the summer months). The runoff from this amount is already included in the mean monthly volume.

The array of the 36 greatest monthly rainfalls as shown on Table A-6 shows that the range between the frequency of the 10 year or a 10 percent chance of occurrence in any one year and the 2 year or a 50 percent chance of occurrence is only 1 inch, (6.7 inches to 5.7 inches). The 3 year or a 33 percent chance is about 5.9 inches of rainfall depth. Referring to Table A-2, the three year frequency occurring for three consecutive days is 2.8 inches, and if, to this, the mean summer monthly rainfall depth of 3 inches is added, the total is 5.8 inches. This depth approaches the 5.9 inches of rainfall depth which has a 33 percent chance of occurrence or has a 3 year frequency. This indicates that the three-year - three consecutive day rainfall plus the normal monthly rainfall would be nearly equal to the expected maximum three-year total rainfall depth. This led to the choice of the three-year - three consecutive day rainfall as the design storm for pollution load computation.

TABLE A-6

36 GREATEST MONTHLY AVERAGE RAINFALLS
PERIOD 1931-1966

Order of Magnitude	Percent of Chance of Occurrence in any one year	All Months	All Months Except Jan., Feb., Nov., Dec.
1	2.8	9.50	9.50
2	5.6	7.01	6.95
3	8.3	6.95	6.61
4	11.1	6.61	6.37
5	13.9	6.58	6.17
6	16.7	6.44	6.07
7	19.4	6.37	6.07
8	22.2	6.17	6.04
9	25.0	6.07	5.90
10	27.5	6.07	5.88
11	30.6	6.04	5.85
12	33.3	5.90	5.81
13		5.88	5.78
14		5.85	5.70
15		5.85	5.59
16		5.81	5.49
17		5.78	5.37
18	50	5.70	5.36
19		5.60	5.21
20		5.59	5.19
21		5.49	5.14
22		5.37	5.14
23		5.36	5.06
24		5.21	5.00
25		5.19	4.87
26		5.16	4.87
27	75	5.14	4.82
28		5.14	4.80
29		5.06	4.76
30		5.02	4.71
31		5.00	4.70
32		4.87	4.69
33		4.87	4.67
34		4.82	4.64
35		4.82	4.60
36	100	4.76	4.60

RUNOFF

In the preceding section, the rainfall data collected were discussed. In this section, we will discuss the method of collection and locations used to collect the runoff characteristics of the study area.

Methods of Collection

All of the major discharge points of pollution to Lake Erie within the study area were gauged. The open streams were gauged with standard float recorders at a site selected with good downstream control to give good low flow control as well as flood flow control. Generally the sites selected were within a walled channel reach. The discharge points that were enclosed in a pipe were gauged with a bubbler type recording gauge.

Instrumentation Used in Streams

Float type gauges were manufactured by Belfort Instrument Company of Baltimore. The gauges were equipped with 48-hour gears, and the charts were changed accordingly. The recorder float was placed in a stilling well of 12 inch corrugated metal pipe, fastened to a wall with iron straps bolted with expansion type anchor bolts in such a position so it would not disturb the flow. The recorder was mounted in a box on top of the pipe fastened with angle iron brackets and was equipped with an eight-day clock. The time was recorded on the chart along with the stage as measured from an independent reference mark at every visit by the sampler. The reference mark was generally a temporary bench mark on the bridge established by survey in reference to the gauge.

Instrumentation Used in Combined Sewer Overflows

The level recorders used in the combined sewer overflows were manufactured by Foxboro and were actuated by either a nitrogen gas bubbler system or closed diaphragm bulb system. The circular charts were changed weekly, and the time was verified at every visit. The depth reading was also verified when there was flow in the storm drain. Since this was not usually the situation, the level recorders and the actuating systems were checked for accuracy twice during the study with a manometer or column of water.

Rating Curve Development in Streams

In order to determine the discharge at various stages as recorded by the depth recorder, it was necessary to develop a rating curve for each gauging site. The streams in an urban area respond quickly to rainfall. The resulting hydrograph usually peaks rapidly and is of a short duration. The hydrograph frequently peaks and returns to low flow in a time period of less than one hour. The runoff water is from the combined sewer system and frequently contains rags and heavy solids. The period of gauging was only six months, and it was necessary to obtain as many velocity measurements at different stages as possible to define the rating curve over a suitable range. Consequently, it was necessary to measure the velocity with methods other than the usual of current meters. Instead, velocity was measured by using a dye. The dye was released nearly instantaneously across the stream by using a plastic bottle with a squirt type nozzle. By measuring the time of passage of the dye field over a measured distance of uniform channel, the average time of passage was computed. The average cross sectional area was measured, and the discharge computed. It is important to use a relatively uniform channel for this method; consequently, each of the streams was investigated, and a uniform reach selected. The method employed allowed the sampler to arrive at a site, check the recorder, make a velocity measurement, measure the stage with reference to the temporary bench mark, take a water quality sample and then leave within ten to fifteen minutes. This simplified method permitted the observation of runoff during short duration storms, and permitted the accumulation of more discharge measurements than would be possible by other methods.

A survey crew took cross sections at each gauging site and referenced the cross section and flow recorder to the temporary bench mark. The cross sectional area could then be determined by measuring the distances from the temporary bench mark to the water surface.

The discharge was plotted against the stage and a rating curve developed for each gauging site.

Rating Curve Development in Combined Sewer Overflows

The combined sewer overflows are natural stream channels which have been enclosed. These combined sewers receive overflow from the combined sewers as well as direct runoff from the street catch basins. These combined sewer overflows have little or no flow except during periods of rainfall. Hydrographs of these overflows are similar to those of the streams. Since the overflows are enclosed, with access only through manholes, measurement of velocity was made by salt concentrations techniques. A section of pipe was selected having a constant slope and size, without any branches entering the pipe between the two manholes chosen for the measurement.

A survey was made to determine the relative elevation of manhole rims and inverts, the distance between manholes and size and slope of the sewer. A bench mark was established by notching the rim for consistency in depth measurements. The distance from the rim to the water surface was measured at both upstream and downstream manholes just prior to and at the end of each test.

A weight with wire and attached electrodes was lowered into the water so the electrodes were at six-tenths depth of flow. Salt was dumped at the upstream manhole from a five gallon pail which was lowered just above the water surface. The timing was started by the person at the downstream manhole when he saw the bucket being tripped. Granulated table salt was used for fast dissolving and was dumped dry. The amount of salt used was approximately one pound per c.f.s. of flow. The passage of the dissolved salt could be accurately noted and timed by reading an ammeter connected to the electrodes in the flow at the downstream station.

The curve of ammeter readings (conductivity) was plotted against time and the centroid of the resulting curve was determined, together with the time from the dumping of the salt to the time of passage of the centroid. The velocity was computed from the time and measured distances.

Results of the velocity measurement permit the determination of Manning's "n" factor. A rating curve was developed for each of the conduits, and extended for high flows using the Manning formula and varying the "n" value for depths of flow, as described by Camp (32). Verification of the variation of "n" value was obtained by some 150 measurements of velocity and Mannings "n" factor in the Cleveland area by Havens and Emerson in prior studies.

Gauging Sites and Results

Continuous gauging of the six streams provided low flow and mean daily discharge data as well as several storm hydrographs from which a unitgraph can be developed. Each of these six streams is discussed in detail. The five storm drains were gauged to provide storm runoff hydrographs from which runoff characteristics of the drainage area could be determined. Each of these five storm drains is also discussed in detail.

DOAN BROOK

Doan Brook drains the western portion of the study area as shown on Figure A-8. The open brook has a continuous flow made up of ground water, overflow from the combined sewer system, and storm runoff. Most of storm runoff reaches the brook by storm or combined sewers. The drainage area, which is primarily residential, is long and narrow with several small lakes in the upper portion. A city park parallels the brook for nearly the entire length. This park includes the Cultural Gardens, the Museum of Art, the Museum of Natural History, and a variety of places of interest.

The brook, when it reaches Lake Erie, is grossly polluted, primarily due to the combined sewer overflows. The brook upstream of the combined sewered area has reasonable good quality typical of a suburban stream. During periods of runoff from storms, the quality deteriorates - BOD, suspended solids, floating solids and debris increase. Doan Brook was gauged and sampled (under a contract with the City of Cleveland) during the summer and fall of 1966, and from February to September 1967 under this contract.

Description of Gauging Site

Doan Brook was gauged and sampled 8,000 feet upstream from its mouth. The drainage area at the gauging station was 6,950 acres. The site was in the walled channel just downstream of Superior Avenue. The channel in this location is regular and uniform, about 35 feet wide and with vertical walls about 5 feet high. The bottom is solid and not subject to change. The depth is uniform across the channel.

The control is created by a concrete slab across the channel about 50 feet downstream of the gauging site. The control did not change during the period of gauging.

DUGWAY BROOK

Dugway Brook drainage area drains the central portion of the study area. Doan Brook borders it on the west, and Nine Mile Creek on the east. This can be seen on Figure A-9.

From Lakeshore Boulevard to Lake Erie, Dugway Brook is an open stream. South of Lakeshore Boulevard the two main branches are both enclosed to the vicinity of Euclid Avenue. South of Euclid Avenue the West Branch is generally enclosed except for short open reaches, and the East Branch is generally open except for short culverted reaches. See Figure A-7. Several small lakes and retention basins in the upper reaches of both branches reduce peak flows. The area north of Mayfield is served by combined sewers whereas the southern suburbs have a separate system. There are 77 known combined sewer overflows in the Dugway system, many of which overflow frequently during dry weather days. Consequently, the brook is grossly polluted. Dugway Brook was sampled and gauged under a contract with the City of Cleveland during the summer and fall of 1966, and from February to September 1967 under this contract.

Description of Gauging Site

Dugway Brook was gauged about 2,000 feet from its mouth at Lake Erie. The drainage area at this point just north of Lakeshore

Boulevard at the end of culverted section is 5,500 acres. The control downstream of the gauge site is a natural rock bottom channel. The sides of the headwall structure extend downstream and contain the width of the channel. The control creates a natural pool allowing good depth discharge relationships.

The higher stages are controlled by the channel and Lake Erie water surface elevations. During this period of the study there was no change in either the low flow or high stage control.

NINE MILE CREEK

The Nine Mile Creek subdistrict drains the central portion of the study area. The Nine Mile Creek subdistrict is shown separately on Figure A-10. The area downstream of Euclid Avenue is served by combined sewers, while in general, the area upstream has separate sewers. The upstream area has a number of cross connections and sanitary regulators which divert raw sewage to the creek during periods of storm runoff.

Description of Gauging Site

Nine Mile Creek was gauged 1,500 feet from its mouth. The drainage area at the gauging site, the downstream side of Lake Shore Boulevard bridge over Nine Mile Creek, is 5,030 acres.

The open channel at this site has a meandering sandy bottom. The right bank has been stabilized by wooden piles. During the period of gauging, a low to medium stage control was established by constructing a low wooden weir. Higher stages are channel controlled. During the period of study, the low weir was washed out and was re-established with no apparent change to the rating. Nine-Mile Creek had the least accurate rating curve.

GREEN CREEK

Most of the Green Creek drainage area is below Euclid Avenue and is in an area served by separate sewers. The area north of Euclid Avenue receives combined sewer overflow, direct industrial wastes - mostly cooling water, and some direct street runoff. The dry weather flow of Green Creek is intercepted at Lake Shore Boulevard and conveyed to the Easterly Wastewater Treatment Plant. Only during occurrences of rainfall does the Creek discharge to Lake Erie. The area south of Euclid Avenue is primarily residential whereas the area to the north of Euclid Avenue is primarily commercial and industrial. (Figure A-11.)

Description of Gauging Site

Green Creek was gauged and sampled at its mouth at Lake Erie. The Creek discharges into the Lake through a concrete outfall structure constructed on the beach at the end of the storm drain. The drainage area at this site is 660 acres. The outfall structure has vertical walls, varies from 8 to 15 feet wide, and is 76 feet long. The end of the structure is frequently submerged by wave action, so a control had to be constructed. A one foot high weir was constructed, about 10 feet from the gauge.

SHAW BROOK

The entire drainage area of Shaw Brook is within the combined sewer area of Cleveland and East Cleveland. The area is primarily industrial and commercial, with scattered residential areas. The Brook receives combined sewer overflows, some direct street runoff, and some industrial waste, which is mostly cooling water. A portion of the dry weather flow of Shaw Brook is intercepted by the Easterly Interceptor, and the remainder goes to Lake Erie. The area is shown on Figure A-12.

Description of Gauging Site

Shaw Brook was gauged about 1,300 feet from its mouth. The gauge was on the upstream side of the culvert that carries the Brook under Lake Shore Boulevard. The drainage area at that point is 680 acres. The control for this gauge was created by building a weir on the upstream side of the culvert. The weir was rated, and the recording gauge established to measure head on the weir. Shaw Brook provided the most accurate data on runoff.

EUCLID CREEK

Euclid Creek is on the east side of the study area. The Creek drains an area of primarily single residence homes with large lots; much of the area is still open land. Figure A-13 shows the drainage area. The entire length of the stream is open. Most of the runoff is from surface runoff or groundwater, except for filter backwash water from the Nottingham Filtration Plant.

Description of Gauging Site

Euclid Creek was gauged 7,800 feet from its mouth. The drainage area at the gauging site is 15,000 acres. This site was chosen because of its exceptionally good control for both low and high flows. The gauge was mounted on a vertical wall about 20 feet upstream from the crest of a 10 foot drop structure constructed for channel erosion control. The concrete drop structure provides an excellent non-changing control for the entire range of flows.

E. 79th STREET OVERFLOW

The East 79th Street combined sewer overflow serves a commercial and multi-family unit residential area. The area contains some of the highest population densities within the Cleveland area. The sanitary sewage from this area is intercepted by the Easterly Interceptor and conveyed to the Easterly Plant. The combined sewer overflow goes to Lake Erie. There is normally no dry weather flow except when an upstream overflow may become plugged, and the depth of the raw sewage flow is above the weir height. During rainfall, the flow increases quickly with the characteristics of urban runoff. See Figure A-14.

Description of Gauging Site

The East 79th Street overflow was gauged and sampled during runoff at a manhole located about 1,000 feet from Lake Erie. The drainage area at this site is 1,490 acres. At this manhole, the lake level causes interference, especially when the wind is from the northeast. A pressure level recorder with a closed diaphragm bulb system was installed in this manhole. The diaphragm bulb was installed in a 6-inch stilling well attached to the wall upstream of the manhole in a location not affected by the lake. The overflow at this location is a rectangular concrete box culvert measuring 16 x 9 feet. The stilling well did not disrupt the flow. The rating studies resulted in determination of a Mannings "n" value of 0.016 for this conduit.

EAST 88th STREET OVERFLOW

The East 88th Street overflow serves the combined sewer system of Cleveland. Dry weather flow is intercepted by the Easterly Interceptor at an overflow structure near the freeway. The overflow goes directly to the Lake through a 72-inch circular brick pipe. The area served is primarily a multi-family unit residential with some commercial business along the main streets. The area is shown on Figure A-15.

Description of Gauging Site

The East 88th Street overflow was gauged and sampled at a manhole just downstream of the overflow structure and about 1,000 feet from the Lake. The pipe controlled the flow at this location. The gauging was done by a bubbler system with the tube suspended near the invert on a rigid wooden frame. The velocity measurements made at this location indicate a Manning's "n" of 0.015.

EAST 156th STREET OVERFLOW

The East 156th Street overflow collects storm water and combined sewer overflow from a residential area on the east side of the study area. The sanitary sewage from this area is intercepted at an overflow structure at Lake Shore Boulevard and is conveyed to the Easterly Plant. The sewer overflow goes directly to Lake Erie. The drainage area at this location is 300 acres and is shown on Figure A-16. There is no dry weather flow at this point.

Description of Gauging Site

The overflow conduit is a 7-foot, 9-inch diameter pipe, which was gauged at a manhole on East 156th Street, about 500 feet from Lake Erie and 300 feet from the overflow structure. The pipe controlled the flow at this site. The depth was recorded by a bubbler type system, with the bubbler tube suspended near the invert on a rigid wooden frame. The velocity measurements indicate a Manning's "n" of 0.015.

NEFF ROAD STORM DRAIN

The Neff Road storm drain serves a single-family unit residential area of 87 acres on the east side of the study area. The sewers in this area are separate, and the Neff Road sewer ostensibly carries only storm water runoff. The 60-inch concrete storm drain empties into the Lake at the end of Neff Road.

Description of Gauging Site

The storm drain was gauged and sampled about 300 feet from Lake Erie. A bubbler system was installed at this site, with the bubbler tube suspended just above the invert by a rigid wooden frame. During this study the storm drain was not successfully rated since the small drainage area and typical short hydrograph did not present a suitable opportunity. A Manning's "n" factor of 0.015 when flowing full was assumed for development of the rating curve. This area and gauging site is shown on Figure A-17.

EASTERLY BYPASS - EAST 140th STREET OVERFLOW

The East 140th Street combined sewer area is just south of the Easterly Plant. The sanitary sewage from this area goes directly to the plant from an overflow in Lake Shore Boulevard, and the combined sewer overflow goes to the plant overflow conduit to Lake Erie. The overflow conduit also receives combined sewer overflow from the large Easterly Interceptor serving the east side of the Metropolitan Cleveland area. The overflow structure for this interceptor occurs just before the sewer reaches the plant, and overflow occurrences are continuously recorded. Since most of the combined sewers are regulated by an overflow before discharging into the Easterly Interceptor, the frequency of overflow is less than a normal overflow. The East 140th Street drainage area is 1,640 acres, and is shown on Figure A-18.

Description of Gauging Site

The Easterly overflow conduit was gauged and sampled about 500 feet from Lake Erie on the 24 x 7-foot concrete box culvert. The depth of flow was recorded on a level recorder with a diaphragm pressure bulb. The diaphragm was enclosed in a 6-inch stilling well fastened to the conduit wall. Velocity measurements at this site indicate a Manning's "n" of 0.015.

DEVELOPMENT OF DESIGN HYDROGRAPHS

Streams

Unitgraphs were developed for Doan Brook, Dugway Brook, Shaw Brook, Nine Mile Creek and Euclid Creek.

The unit period for all streams was 15 minutes, and in all cases, 15 minutes was less than the basin lag time. The base flow was consistently removed by drawing a straight line from the beginning of the abrupt rise of the hydrograph to the break in the slope of the recession curve. The volume of water was measured by planimetering the hydrograph and converting to inches of runoff. This was compared to the total depth of rainfall. The total amount of infiltration and other losses is the difference between the two depths. In order to determine the excess rainfall by unit periods, this total loss must be proportioned over the entire rainfall period. The period of gauging was short, and the available hydrographs were generally from low rainfall rates. Consequently, most of the runoff was attributed generally to impervious areas, and rainfall on pervious areas was assumed lost by infiltration, since the intensity of storms investigated was much less than the infiltration capacity. (Infiltration capacity curve was assumed as average between "Residential Standard Curve" and "Reduced Industrial and Commercial Curve".)(33) In more intensive storms, pervious areas would contribute to runoff and the excess rainfall from these areas was assumed as the total rainfall minus infiltration capacity. The general term of excess rainfall was:

$$DE = C \times \text{Imp. Ratio} \times D_T + \text{Perv. Ratio} \times (D_T - D_L)$$

where: D_E = Excess depth of rainfall (that would runoff)
C = Coefficient of runoff from impervious area which accounts for losses by depression storage and discharges to pervious areas that is eventually lost by infiltration. This coefficient is based on computations from gauged hydrographs.

Imp. Ratio = Ratio of impervious area to total area of the district. A detailed estimate of the impervious area was made based on aerial photographs.

D_T = Total depth of rainfall in 15 minutes duration (which is the unit hydrograph duration)

Pervious Ratio = 1 - impervious ratio

D_L = Depth of rainfall lost by infiltration. It depends on both the intensity and duration of antecedent rainfall. An equation was developed to account for these variables.

Once the selected hydrograph and its corresponding excess rainfall was known, the Collin's method (34) was used to obtain a trial unitgraph. In many cases storms were selected that had a single unit period of excess rainfall. The trial unitgraph was used to obtain computed hydrographs from recorded rainfall for comparison with the actual gauged hydrographs. Two or three adjustments were generally required to arrive at a final unit hydrograph.

Design storm hydrographs were developed for a frequency of occurrence of 1, 3 and 5 years. The rainfall used was the most intensive 14 periods of the 1, 3 and 5 year design rainfall data. The excess rainfall was computed for every period of rainfall. Using the excess rainfall per 15 minute period and the final unit hydrograph, the design storm hydrograph was produced by adding runoff rates contributed to each period by the rainfall excess.

All these computations and the previous trials were greatly simplified by the application of a computer program to perform all the steps previously mentioned. The input required for the program was: the unit hydrograph, number of rainfall periods, and depths of rain in every period. The output was the ordinates of the hydrograph at 15-minute intervals.

The hydrographs of 1-year, 3-year and 5-year design storms are shown on Figures A-19 through A-23.

COMBINED SEWER OVERFLOWS

The storm drain runoff characteristics were measured. Percentage of runoff, basin lag and the Rational Method "C" factor were measured from the gauged hydrograph. Time of concentration and imperviousness was measured or computed from maps or aerial photographs. Table 2-1 gives this information.

In order to compute hydrographs for the storm drains, a modified Chicago Hydrograph method was used. Sufficient data were not available to develop unitgraphs, and synthetic methods did not produce reasonable results. The modified Chicago Hydrograph method was developed for use in Cleveland, based primarily upon Tholin and Keifer's paper(35) as a part of the master plan study already mentioned. This method was applied to East 72nd Street, East 88th Street, East 156th Street, Easterly bypass-East 140th Street, Neff Road, and Green Creek. Using the design storm rainfall, hydrographs of the 1, 3 and 5-year storms were produced. It was necessary to produce hydrographs for these storm drains rather than only computing the peak discharge because some of the sites involve pumping and pumping stations, and secondly in order to minimize the collection system pipe size, the hydrographs were routed to gain the benefit of time offsetting of the hydrographs. These computed hydrographs are shown on Figures A-24 and A-25.

TABLE A-7

SUMMARY OF ANALYSES

LOCATION	TOTAL N mg/l	TOTAL PO4 mg/l	SUSPENDED SOLIDS mg/l	COD mg/l	BOD mg/l	TOTAL COLI /100 ml	FECAL COLI /100 ml	FECAL STREP. /100 ml
DOAN BROOK								
Number of Analyses	31	50	48	31	49	31	50	31
Minimum	0.0	0.10	0.4	12	0.1			
Maximum	11.2	8.40	238	136	42.8			
Average	3.2	1.55	68	68	12.4	24.3×10^5	2.6×10^5	14.7×10^3
Median	2.5	2.14	52	65	9.0			
DUGWAY BROOK								
Number of Analyses	32	52	51	33	52	32	52	32
Minimum	3.1	1.30	2	24	7.3			
Maximum	15.3	14.50	164	224	81.0			
Average	7.8	5.58	41	106	23.7	14.3×10^6	1.8×10^6	1.4×10^5
Median	7.2	4.55	30	92	21.3			
NINE MILE CREEK								
Number of Analyses	33	52	50	33	51	33	52	33
Minimum	1.4	0.60	4	24	2			
Maximum	8.8	9.00	339	212	52.5			
Average	4.1	3.71	53	80	13.7	5.8×10^5	1.8×10^5	1.2×10^3
Median	3.9	3.25	27	64	11.1			
SHAW BROOK								
Number of Analyses	32	52	50	32	52	32	52	32
Minimum	0.3	0.10	1	8	2.4			
Maximum	14.8	30.50	319	272	85			
Average	2.7	5.38	46	75	12.0	1.13×10^6	1.12×10^5	2.9×10^3
Median	1.7	2.95	32	56	7.0			
GREEN CREEK								
Number of Analyses	14	16	16	14	17	14	16	14
Minimum	0.4	0.10	10	20	3			
Maximum	4.2	4.80	460	380	80.3			
Average	2.4	1.74	165	114	20.8	1.6×10^6	1.6×10^5	3.6×10^4
Median	2.3	1.30	130	93	11.7			
EUCLID CREEK								
Number of Analyses	32	51	47	30	50	32	51	32
Minimum	0.0	0.10	5	4	1			
Maximum	9.7	5.70	134	130	53			
Average	1.9	1.30	78	54	7.8	8×10^5	1.8×10^5	7×10^3
Median	1.5	1.10	73	51	5.4			

TABLE A-8

SUMMARY OF ANALYSES

LOCATION	TOTAL N mg/l	TOTAL PO4 mg/l	SUSPENDED SOLIDS mg/l	COD mg/l	BOD mg/l	TOTAL COLI /100 ml	FECAL COLI /100 ml	FECAL STREP. /100 ml
E.79th STREET - COMBINED SEWER OUTFLOW								
Number of Analyses	5	5	5	5	5	5	5	5
Minimum	6.0	4.40	153	268	57			
Maximum	18.6	18.50	297	840	290			
Average	10.7	9.00	225	505	148.4	35 X 10^{6}	4.8 X 10^{6}	3.8 X 10^{5}
Median	8.8	6.80	211	384	81.5			
E.88th STREET - COMBINED SEWER OUTFLOW								
Number of Analyses	7	8	8	7	8	7	8	7
Minimum	6.0	2.50	34	100	29.2			
Maximum	19.2	26.50	217	344	98.3			
Average	12.2	9.81	121	249	57.4	27 X 10^{7}	9.5 X 10^{6}	3.8 X 10^{5}
Median	10.8	6.55	114	260	52.4			
E.156th STREET - COMBINED SEWER OUTFLOW								
Number of Analyses	11	13	13	11	14	10	13	11
Minimum	4.2	1.20	37	80	12			
Maximum	11.0	11.00	272	480	115			
Average	6.4	5.20	100	194	40.8	5.1 X 10^{6}	2.0 X 10^{6}	7.9 X 10^{4}
Median	6.2	4.80	85	140	31.2			
NEFF ROAD - STORM DRAIN								
Number of Analyses	15	27	20	15	28	15	27	14
Minimum	0.0	0.10	2	16	1.3			
Maximum	3.8	12.50	110	184	60			
Average	1.7	1.35	40	78	11.9	16 X 10^{5}	33 X 10^{4}	14 X 10^{3}
Median	0.8	0.50	33	64	7.4			
EASTERLY - S.T.P. OVERFLOW								
Number of Analyses	12	14	14	12	14	12	14	12
Minimum	7.0	2.60	97	180	32.5			
Maximum	18.6	21.00	306	656	115.5			
Average	13.2	8.54	156	316	63.8	12.8 X 10^{6}	2.2 X 10^{6}	1.7 X 10^{5}
Median	13.1	7.65	126	310	53.1			

APPENDIX B

PRELIMINARY STRUCTURAL DESIGN

General

The basin is to be constructed in water varying in depth from 10 to 45 feet. About 60 percent of the wall length will be built in water deeper than 40 feet. The bottom of Lake Erie in this area consists of various clays and silty clays on top of glacial till, with a thin layer of mud or sand overlying this material.

At the present time, there are very limited records available regarding the exact composition or thickness of these layers or their bearing capacity. The data available indicate that the foundation materials are soft and have a limited bearing capacity, and for preliminary design purposes it has been assumed necessary to provide foundation material in a dredged channel.

Wave Forces

Waves are generated by winds. Their characteristics can be determined by the velocity of the wind, its duration and the fetch length. The sizes of various waves are determined either by an analysis of synoptic weather charts or by direct observations.

Structures may be subjected to three types of wave action - breaking, broken and nonbreaking. Waves moving in depths greater than half their length are deep-water or non-breaking waves and are unaffected by the water depth. Forces exerted by non-breaking waves are essentially hydrostatic, due to the rise and fall of the standing wave.

As the deep-water waves move in toward the shoreline their characteristics are modified by the decreasing depths of water. That part of the wave crest advancing in shallower water moves more slowly than that part still advancing in deeper water; this causes the wave crest to refract toward alignment with the underwater contours. Refraction changes the direction and intensity of wave attack on the beach or shore-protection structure. At a certain point in its advance toward shore, a wave becomes unstable, peaks up and breaks. The water motion changes from a laminar orbital motion to turbulent, white-water condition.

The proposed wall has been designed to resist the action of the non-breaking wave, since in deeper water, the breaking wave cannot occur. The design was checked for the forces exerted by a breaking wave in shallower water, and it was found that the non-breaking wave governed the design. Broken waves reaching the structure in shallow water have a lesser effect.

The U.S. Corps of Engineers has presented the best available data on determination of waves generated in Lake Erie(36). Based on these data, a wave of 17 foot height having a length of 128 feet and an expected frequency of occurrence of once in 25-years was assumed for design. Higher waves would have only slight additional effect, since the freeboard of the wall would be exceeded.

Non-breaking waves form a clapotis or standing wave in front of the structure; hence the forces exerted are essentially hydrostatic. To compute the maximum and minimum wave pressures for the non-breaking wave, the procedure of Sainflow(37) was used.

The breaking wave exerts a combination of hydrostatic and dynamic forces on the wall. These forces were computed in accordance with the procedure proposed by Minikin in the U.S. Corps of Engineers Technical Report(37), for checking the breaking wave forces in shallower water.

Records of the U.S. Weather Bureau show that waves of the general magnitude of the design wave can be expected at Cleveland from a sector of 145°. For this reason, all three of the exposed sides of the basin have been designed for the same conditions.

Besides the wave forces, a breakwater wall is subject to ice pressures. A precise analytical method of determining ice forces on breakwaters is not known. However, Striegl(38) refers to allowances on the Great Lakes for ice pressure ranging from zero to 50,000 pounds per lineal foot. Normally, design of breakwaters on the Great Lakes is governed by wave action, since the pressures generated by waves where the fetch is long probably exceed the maximum pressure developed by an ice sheet, and since the two forces do not take place simultaneously. For purposes of this study, ice pressure was assumed to be less than wave action, and was not governing. The design adopted to resist wave action was checked for resistance to ice pressure, and is sufficient to resist a maximum ice pressure of 35,000 pounds per lineal foot, which is considered an adequate allowance for preliminary purposes.

Type of Wall

Consideration was given to three general types of wall construction: a rubble-mound stone breakwall; a combination wall of rubble base and pre-cast concrete cells filled with stone; and a filled cellular sheet piling cofferdam. The stone breakwall was rejected on the basis of excessive cost and unavailability of material; investigation revealed that an adequate source of stone is not available near Cleveland for the very large quantities required for this deep water project, and costs of shipping stone in these quantities required were excessive.

A preliminary study of the pre-cast concrete rubble-filled caissons indicated that this alternative was not economical. Dry docks large enough for construction of the caissons are not available in Cleveland, and manufacture and towing of the caissons from the nearest available facility in Lorain,Ohio, would probably require several construction seasons and substantial risk in transportation. In addition, the relatively poor bearing value of the foundation material would require a rubble base for the cells of such volume that again local quarry resources would be inadequate, and the economies of pre-cast construction would be partly offset by the necessity of constructing the more massive foundation in situ.

The most feasible solution was found to be a cellular steel sheet pile cofferdam. This type of wall has the advantage of a vertical wall which requires less freeboard to reduce overtopping, and is superior from the standpoint of operation of the basin. Life expectancy exceeds 50 years, and maintenance of this type of structure is minimal. This type is also less permeable to water movement than the other types contemplated, which is advantageous in this case. Finally, the cellular cofferdam wall was estimated to have the lowest construction cost of the three alternatives. The material to be used for filling the cofferdam cells was assumed to be sand dredged from deposits offshore of Fairport Harbor. This is subject to revision in final design, since the weight and angle of internal friction required of the fill material, and the diameter of the cell are affected by the final bearing value of the foundation material.

Water Level and Freeboard

Mean water level of Lake Erie at Cleveland is Elevation 572.31. (New York 1935 MSL datum). Statistics on seasonal variations indicate that mean level during the more critical summer period is about 1 foot higher. Therefore, design was based on a water level of 573.3. Since Cleveland is located in a nodal zone of Lake Erie, no seiche effect has been considered in establishing water level.

The basin freeboard is an important design consideration. Although a high freeboard is desirable to prevent displacement of basin contents by overtopping waves, the effect of freeboard on structural design is substantial, and high freeboard is costly. Alternative designs were prepared for three different freeboard heights, which are presented in the next section.

Design of Wall

Alternate designs were prepared for various freeboard heights and for different design waves. Results of the preliminary designs are shown in Table B-1.

The recommended preliminary design is as follows:

Type of Wall - Steel sheet piling cellular cofferdam, sand filled.

Freeboard - 10 feet above design water level (Top Elev. 583.3)

Cell diameter - 67 feet, approximately.

Cell spacing - 72 feet, center to center of circular cells.

Cap - concrete or asphaltic concrete, 24 inches thick.

TABLE B-1

ALTERNATIVE PRELIMINARY DESIGN DATA

Case No.	Freeboard, Ft. Above Design W.L.*	Design Wave Data: Height ft.	Design Wave Data: Length ft.	Design Wave Data: Period sec.	Maximum - Overturning Moment Ft. kips	Cofferdam Cell Dia. - ft.	Overtopping Wave Height ft.	Bearing on Foundation lbs/sf
I	10	17	128	5	1,562	67	7	6,280
			185	6	1,403	65		6,450
II	13.4	17	128	5	1,746	67	8	7,200
			185	6	1,570	64		7,250
III	25.1	17	128	5	2,158	67	14	9,160
			185	6	1,818	67		8,620

*Design W.L. = 573.3 1935 MSL Datum.

APPENDIX C

COLLECTION SYSTEM DESIGN

Some of the details of the hydrograph routing and a more detailed description of the system is given herein. The hydrographs as developed were routed by the time-offset method. The hydrograph resulting from an input point upstream was shifted to the right on the time scale a distance (in time) equal to the travel time between the upstream input point and the point under consideration. The time of travel of the peak was calculated by Manning's formula.

The shifted hydrograph was superimposed on the hydrograph joining the collector at that point, and the addition of the two hydrographs yield the combined hydrograph at the point under consideration. Attenuation in the gravity portions of the collection system was not considered due to the short travel time.

WEST COLLECTION SYSTEM AND EASTERLY PLANT

E. 79th Street - Doan Brook

This branch of the collection system starts at E. 79th Street just north of Easterly Interceptor. The design flow entering the collection system is 900 cfs, and an overflow structure will divert excess flows to the existing pipe which empties into Lake Erie. The new pipe is 102 inch R.C.P. for 1,000 feet, and 96 inch R.C.P. for 1,000 feet.

Doan Brook - E. 88th Street

The hydrographs of E. 88th Street and Doan Brook give a combined hydrograph with a peak discharge of 1,350 cfs. An overflow structure will be built just south of Lake Shore Boulevard at Doan Brook culvert to take a total flow of 1,350 cfs to the new system. Excess flows will be discharged to Lake Erie through the existing Doan Brook culvert. The conduit between Doan Brook and E. 88th Street is a twin concrete box 11.0 x 10.2 feet designed for a discharge capacity of 1,350 cfs.

E. 88th Street - Dugway Brook

The hydrograph of E. 88th Street was combined with the hydrograph of E. 79th Street and Doan Brook to obtain the total hydrograph shown on Figure C-1. A pumping station is planned at this point. The distance between E. 88th Street and Dugway Brook was 6,400 feet.

In order to reduce the pumping capacity required, it is planned to construct a holding basin to accept flows exceeding a peak pumping rate of 500 cfs. At total flow rates in excess of 500 cfs, flow will be diverted to the holding basin until storm flow drops again to this level. After flows diminish, the pumps will dewater the basin. The volume of storm runoff above the 500 cfs level is 2.3 million cubic feet which requires a holding basin area of 115,000 square feet at a depth of 20.0 feet.

A trash rack will be required ahead of the pumping station to remove debris larger than 3 inches. The force main will be a reinforced concrete pipe 96 inches in diameter with a total length of 6,400 feet.

Dugway Brook - Stabilization Basin

The peak flow at Dugway Brook is 1,880 cfs, the sum of 1,380 cfs from the Brook and 500 cfs from the pumping station. An overflow will be required to discharge flows greater than 1,380 cfs from Dugway Brook. A trash rack will be installed at the junction point.

The flow will be carried through a concrete open channel 31.3 x 12.0 feet for 250 feet, beyond which two 144-inch pipes will be used. The pipes will slope down as shown in profile on Figure 6-1, to provide ample draft for boats using the passage way to the enclosed bay and marina.

Nine Mile Creek - Shaw Brook

The one-year design peak flow of Nine Mile Creek is 1,130 cfs. An overflow will be built to divert excess flows to the Lake after chlorination. The flow will be carried through a twin concrete box conduit (10.0 x 9.4 feet) for a distance of 700 feet to the Shaw Brook input point.

Shaw Brook - Stabilization Basin

The design flow at this point is 1,250 cfs, the peak discharge of the combined hydrograph of Nine Mile Creek and Shaw Brook. Excess flows will be chlorinated and diverted to the Lake through an overflow.

A trash rack will be built downstream of this point to collect debris larger than 3 inches. From this point the design flow is carried through a concrete open channel (20.7 x 12.0 feet) about 250 feet long. From this point to the basin, two 120-inch reinforced concrete submerged conduits will be used to provide boat access to the bay.

Easterly Plant Outfall - Basin

The design flow of the Easterly plant outfall is the maximum flow expected in the year 2000 (2-1/2 times the daily average), which is 586 cfs (378 million gallons daily). This flow will be carried through two 108 inch reinforced concrete pressure pipes from the end of the existing effluent pipes to the stabilization basin, a distance of about 350 feet. The conduits should be laid in trench to provide draft for boats using the passageway between the bay and the Lake.

It is not possible to discharge the maximum flow to the basin at the high Lake levels without flooding the final settling tanks. Therefore, a pumping station will be required to pump the final settling tanks flow (352 cfs). Pumping would be required during the time the Lake level is above 572+. Flow from the primary tanks would not require pumping.

Easterly Plant Overflow - Basin

The design flow used for the overflow is 1,400 cfs, which is equal to the capacity of the existing conduit. Two 120-inch reinforced concrete pipes will be required from the end of the existing conduit to the stabilization basin.

EAST COLLECTION SYSTEM

This system collects the runoff from the area between Green Creek and Neff Road.

Neff Road to Euclid Creek

The design flow is the peak discharge of Neff Road hydrograph which is 61 cfs. An overflow will be built to divert excess flow to the Lake through the existing pipe. A 42-inch pipe about 980 feet in length will be required.

Euclid Creek - Green Creek

The combined hydrograph of Neff Road and Euclid Creek is shown in Figure C-2. The peak discharge is 1,867 cfs. It will be necessary to pump the flow to the basin because of the low head available and the long distance (7,740 feet) from Euclid Creek to the basin.

The pumping capacity was reduced from the peak discharge of 1,867 cfs to a level of 800 cfs by diversion of excess storm flows to a holding basin as shown in Figure 6-2. After the storm flow has receded to a rate below 800 cfs, the pumps will empty the holding basin. The required volume of the basin is 6.3 million cubic feet, and at a 20 foot depth, the holding basin would be about 300 feet by 1,050 feet in plan. The north side of the basin will be a low earth dam with a spillway section to discharge excess flow to the Lake. The spillway crest will be at elevation 575.

The flow to the pumping station will be carried through a diversion channel from the south end of the holding basin to the pumping station. This channel would have invert elevation of 569 at the diversion point and water surface at 575 when flowing at a capacity flow of 800 cfs. The function of the channel is to reduce the pumping head by routing flows around the holding basin.

The discharge line from the pumping station to Green Creek is a 120-inch reinforced concrete pipe 3,290 feet long.

Green Creek - Stabilization Basin

The flows from E. 156th Street and Green Creek are combined at the intersection of Lake Shore Boulevard and E. 156th Street. The one-year flow of 600 cfs will be carried through the existing Green Creek culvert and a new supplementary pipe running parallel to it. The new pipe would be 54 inch diameter. An overflow would be built at the crossing of E. 156th Street and Green Creek conduit to divert flows higher than 600 cfs to the Lake through the existing E. 156th Street storm drain. (Figure C-3).

A pumping station is required for this combined flow, and its discharge line will join the force main from Euclid Creek. The pumping station capacity will be 400 cfs, and higher flows will be diverted to a holding basin. The volume of the holding basin is 252,000 cubic feet. By using a depth of 20 feet, the area required will be 100 feet x 126 feet.

The 120-inch force main from Green Creek to the stabilization basin, will serve as a common pipe for both Euclid Creek and Green Creek pumping stations for a distance of 4,250 feet to the basin.

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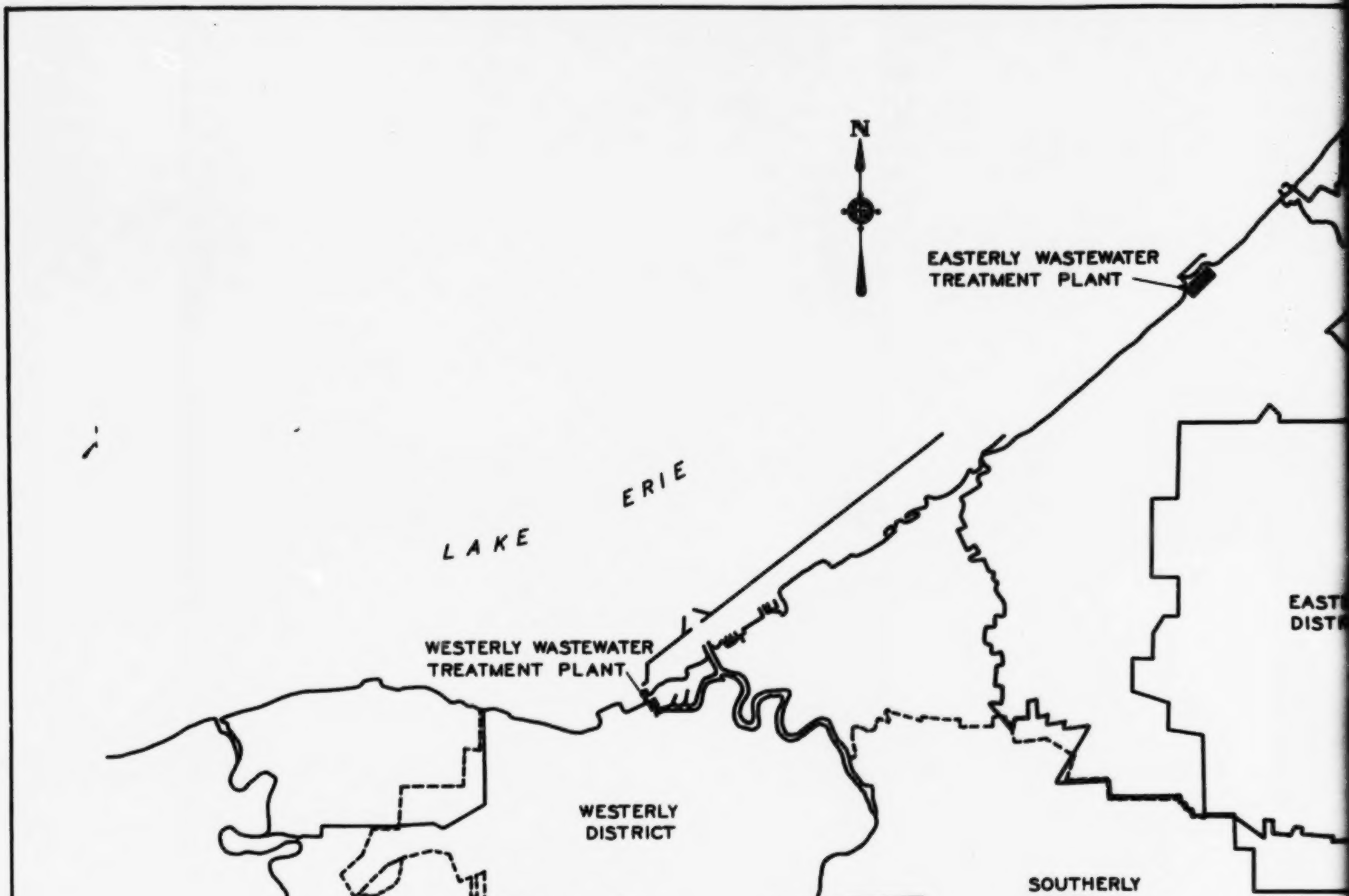
FIGURES


N
EASTERLY WASTEWATER TREATMENT PLANT
LAKE ERIE
EAST
DIST
WESTERLY WASTEWATER TREATMENT PLANT
WESTERLY DISTRICT
SOUTHERLY DISTRICT
CITY OF CLEVELAND CORPORATION LINE
SOUTHERLY DISTRICT
SOUTHERLY WASTEWATER TREATMENT PLANT
LIMIT OF METROPOLITAN CLEVELAND SEWERAGE DISTRICT
HAVENS AND EMERSON

A

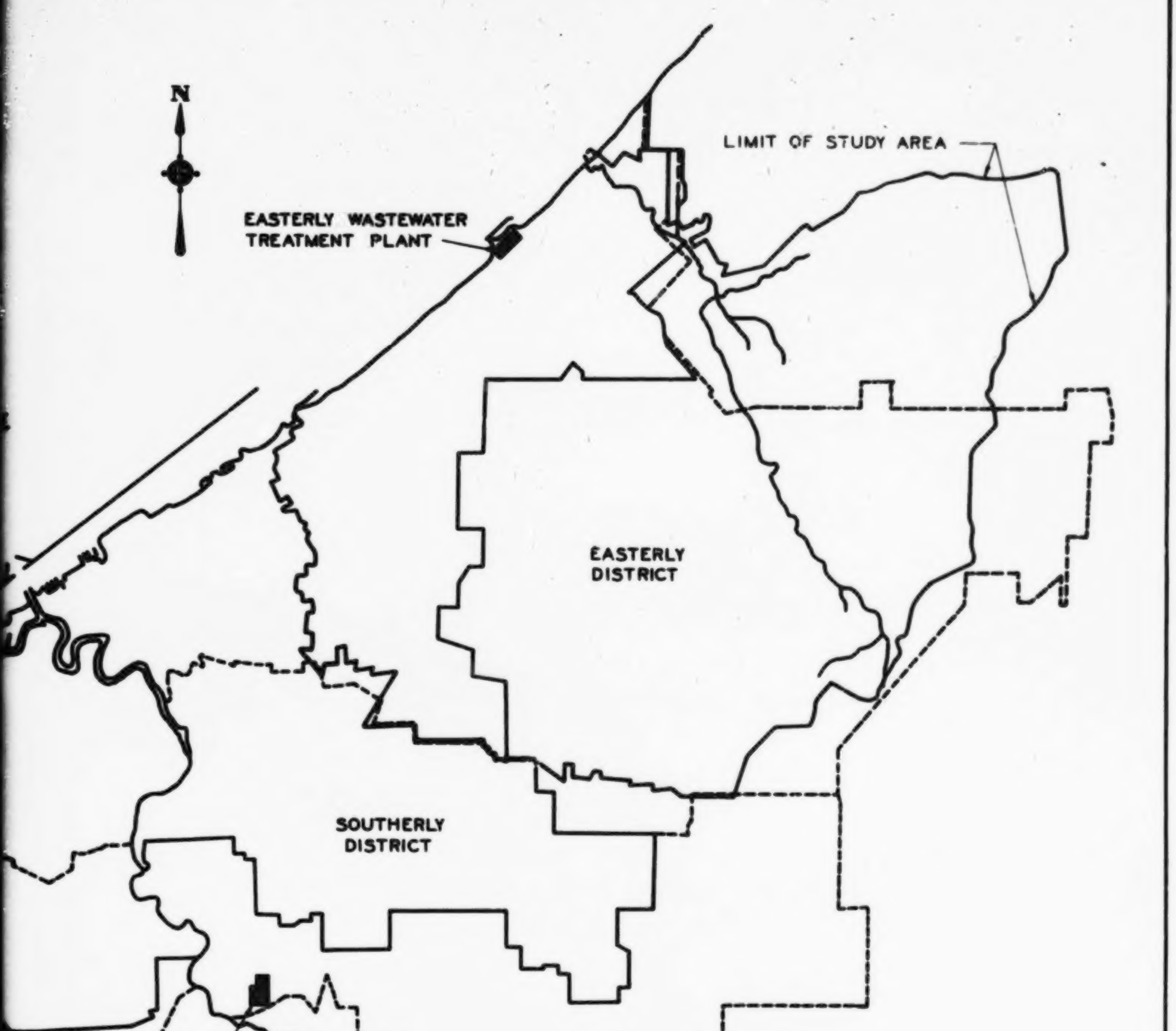

N
LIMIT OF STUDY AREA
EASTERLY WASTEWATER TREATMENT PLANT
EASTERLY DISTRICT
SOUTHERLY DISTRICT
SOUTHERLY WASTEWATER TREATMENT PLANT
LIMIT OF METROPOLITAN CLEVELAND SEWERAGE DISTRICT

FIGURE 2-1
FEASIBILITY STUDY OF
STABILIZATION-RETENTION BASIN
MAP OF STUDY AREA

B

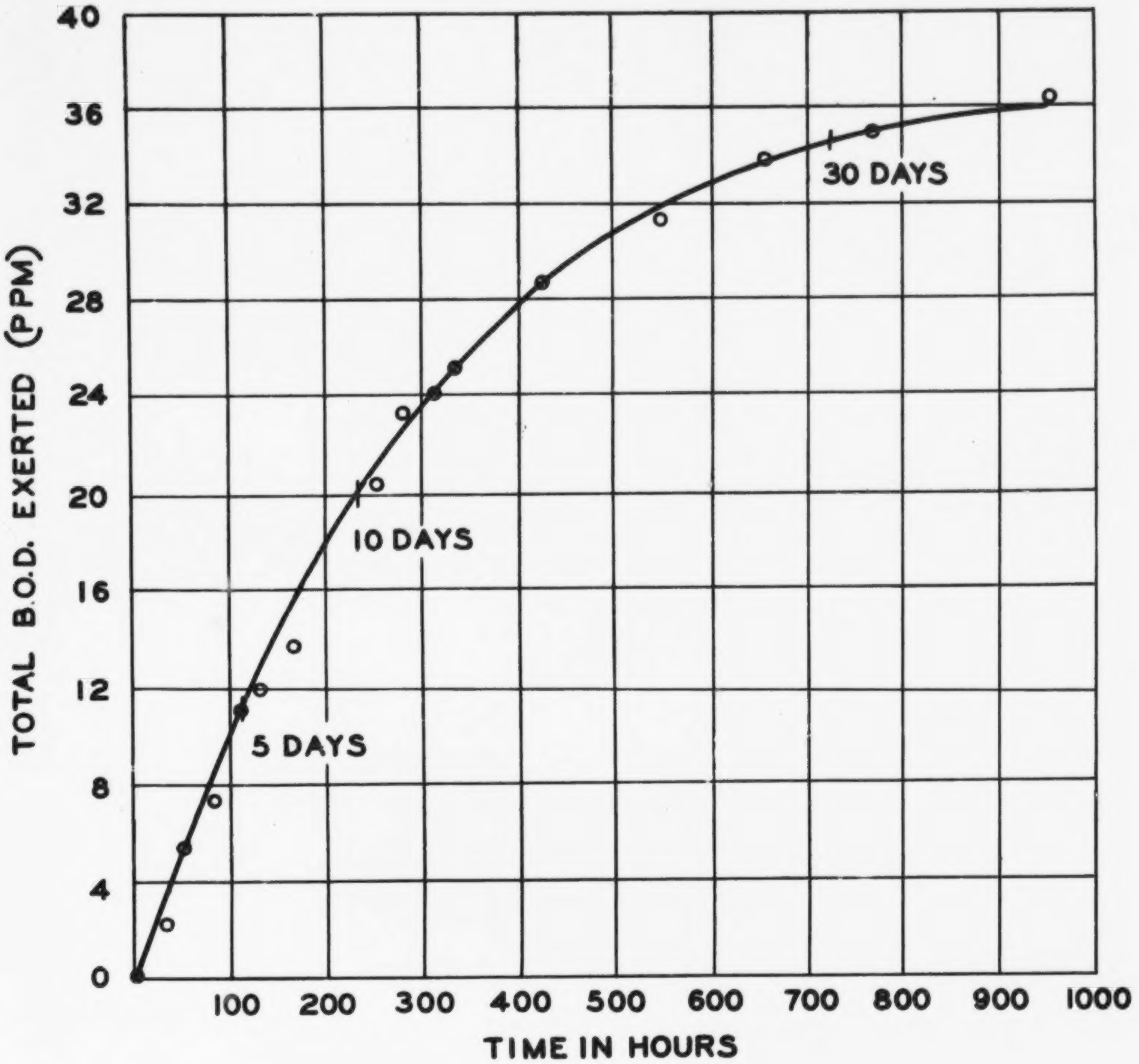


FIGURE 4-1
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN

LONG TERM B.O.D. EASTERLY WASTEWATER PLANT EFFLUENT

HAVENS AND EMERSON

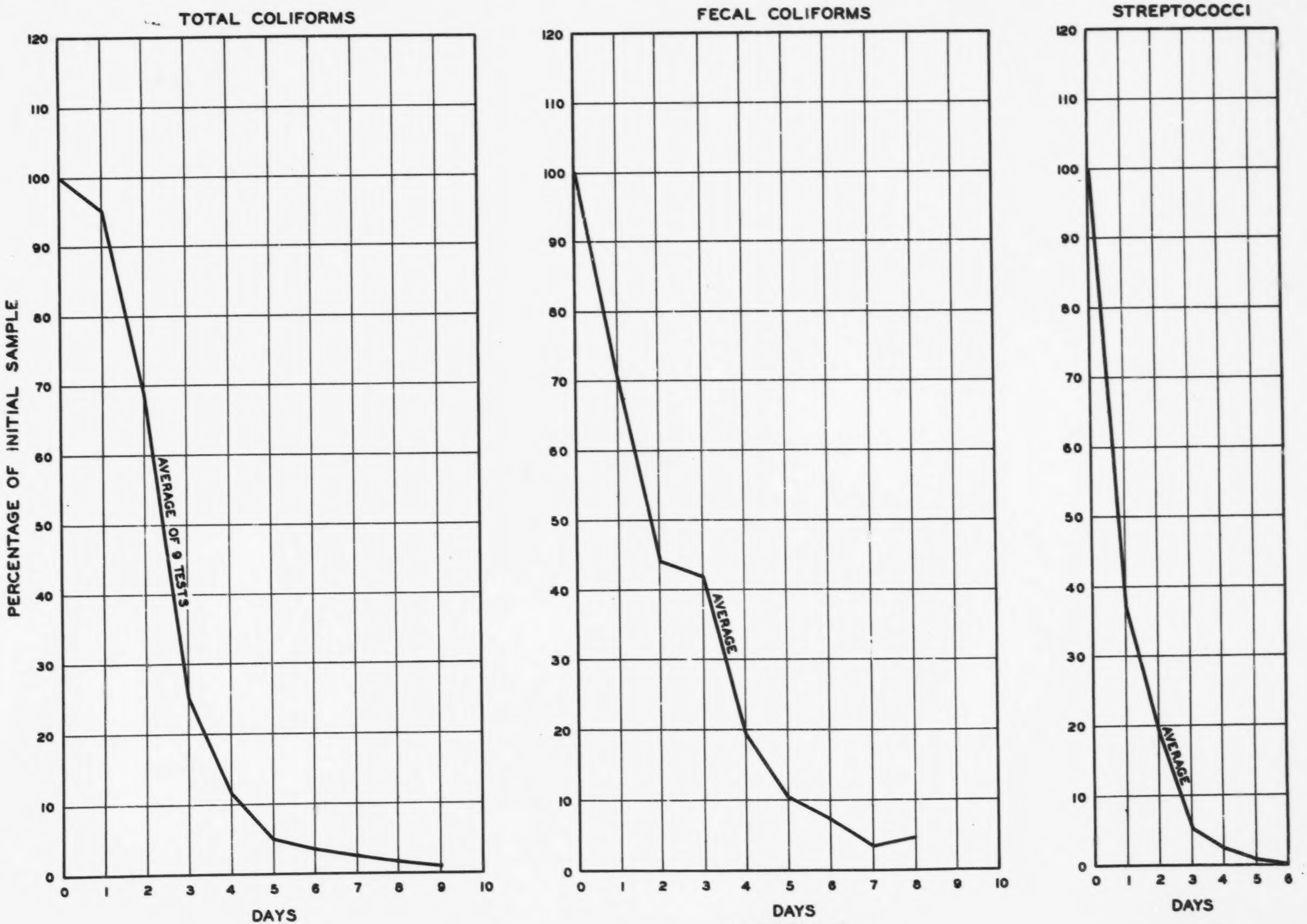
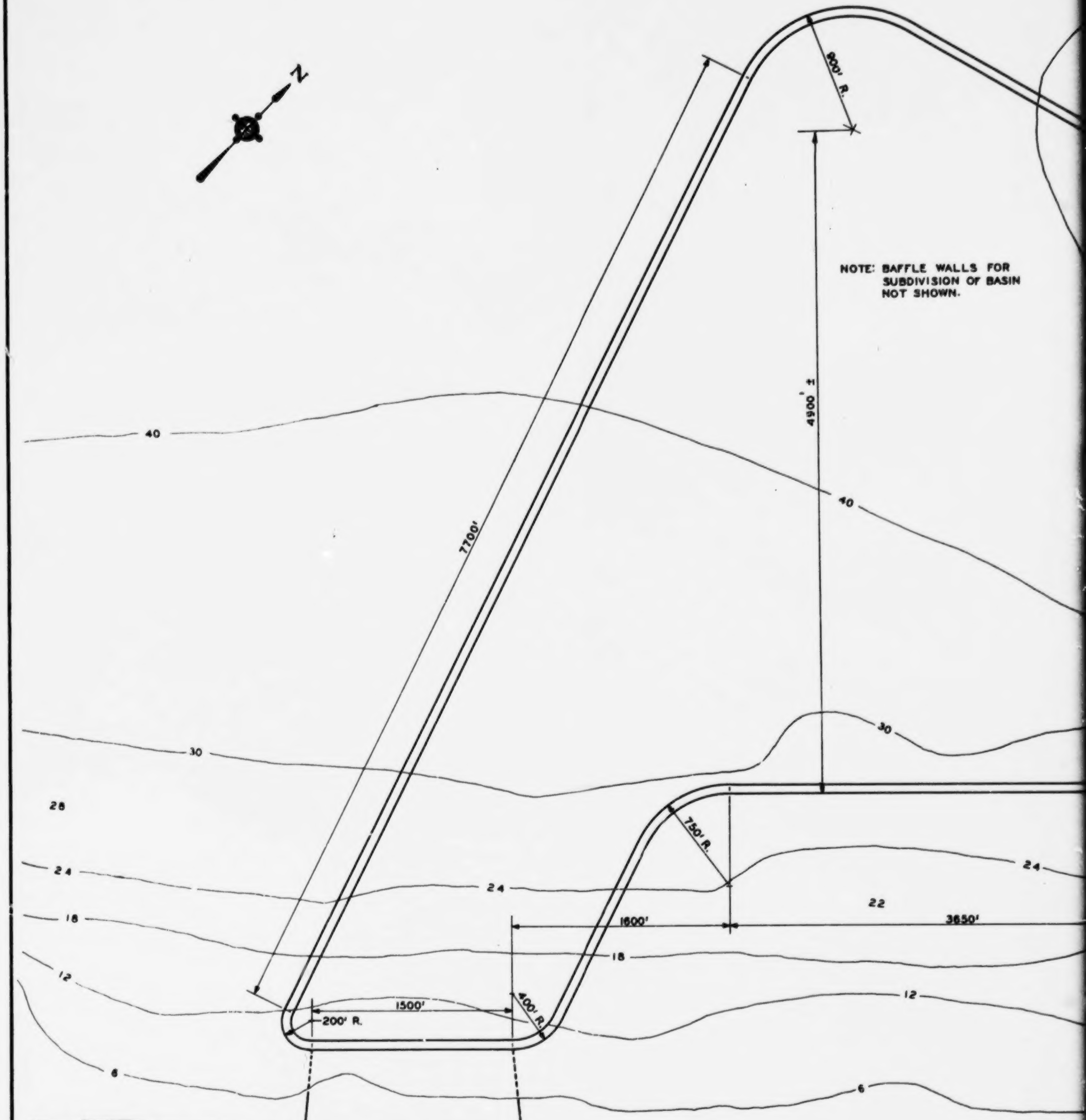


FIGURE 4-2
VIABILITY OF BACTERIA
IN LAKE ERIE WATER

HAVENS AND EMERSON


N
NOTE: BAFFLE WALLS FOR SUBDIVISION OF BASIN NOT SHOWN.
800' R.
4900' ±
7700'
750' R.
1600'
3650'
1500'
200' R.
400' R.
40
40
30
30
28
24
24
24
22
18
18
12
12
6
6
FOR DETAILS OF THE WEST COLLECTION SYSTEM, SEE FIG. G-1
DUGWAY BR.
SHAW BR.
NINE MILE CR.
FEET
0
500
1000
1500

HAVENS AND EMERSON

A

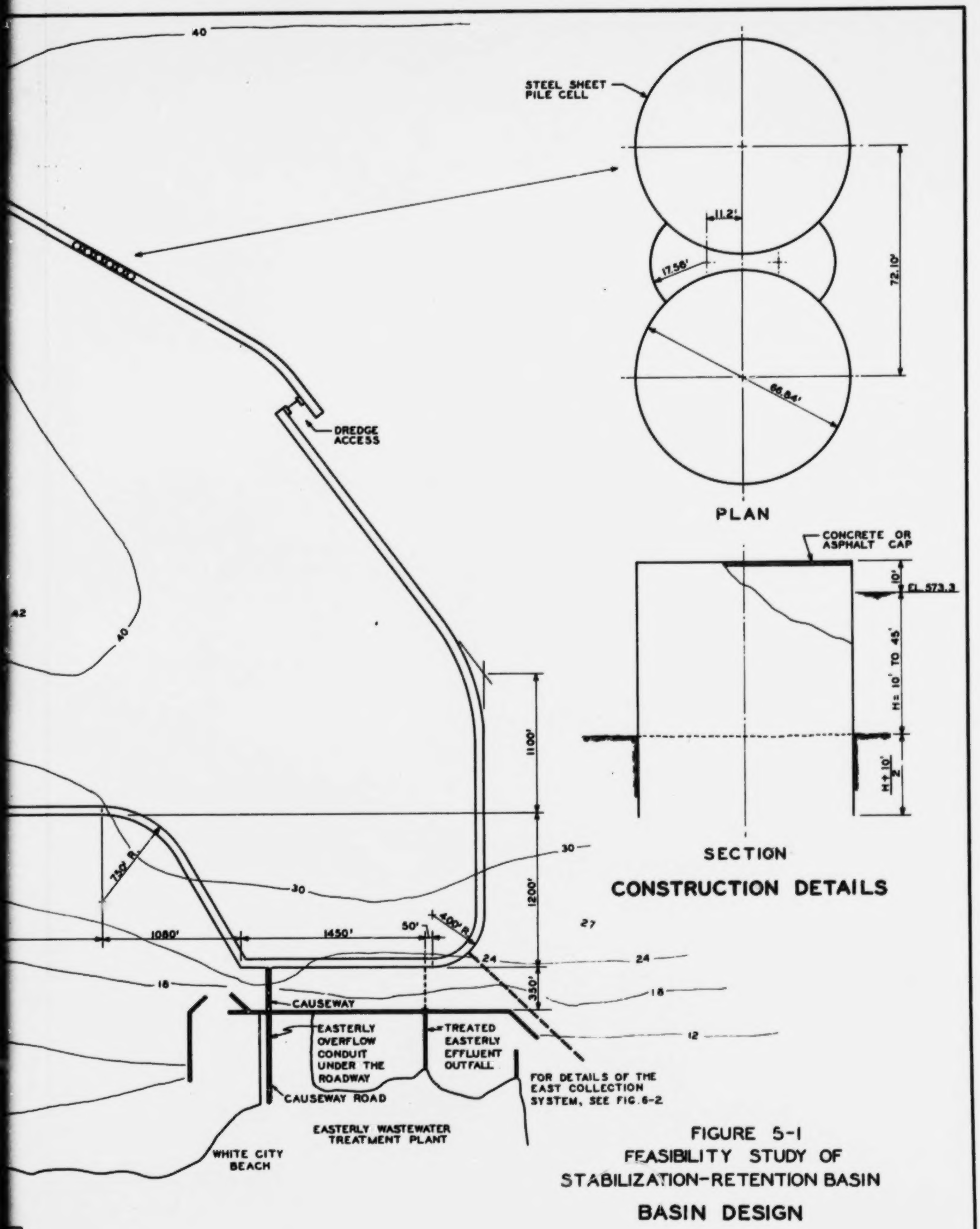
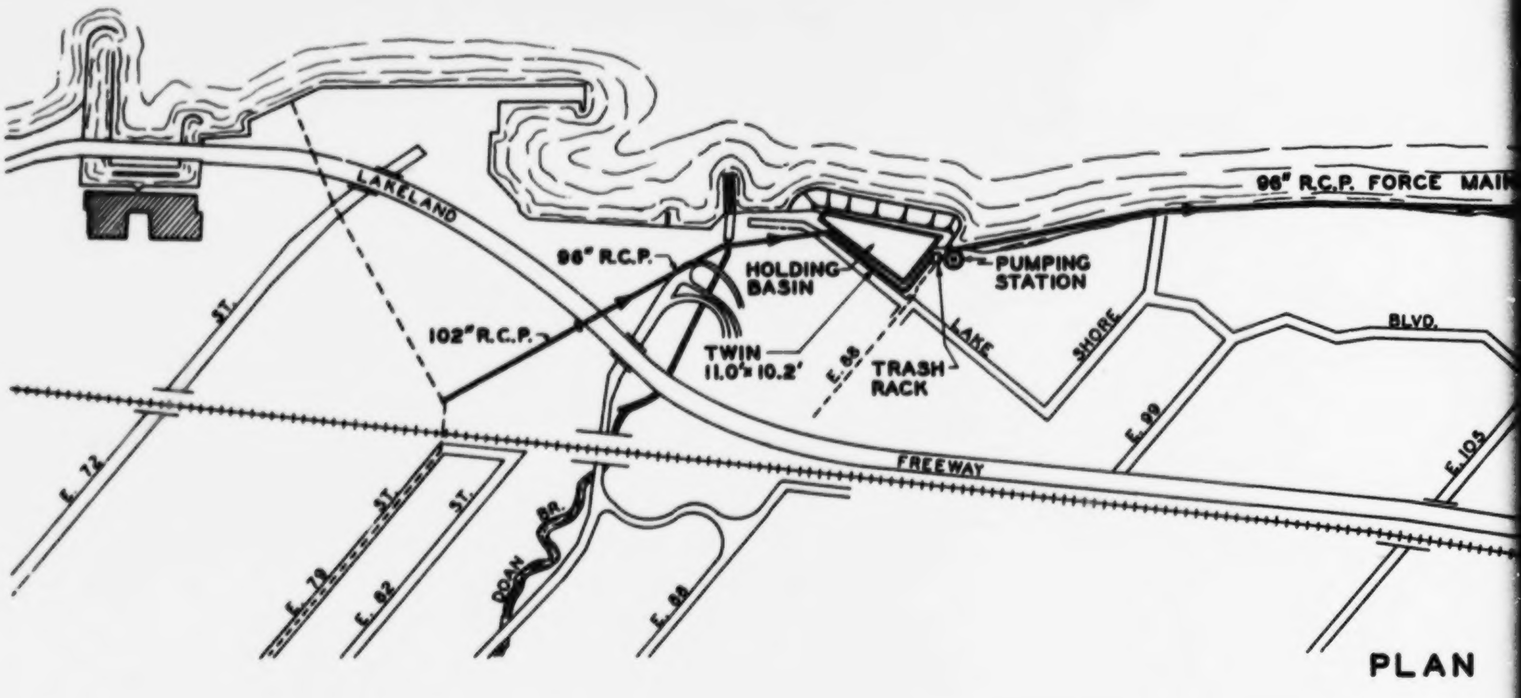
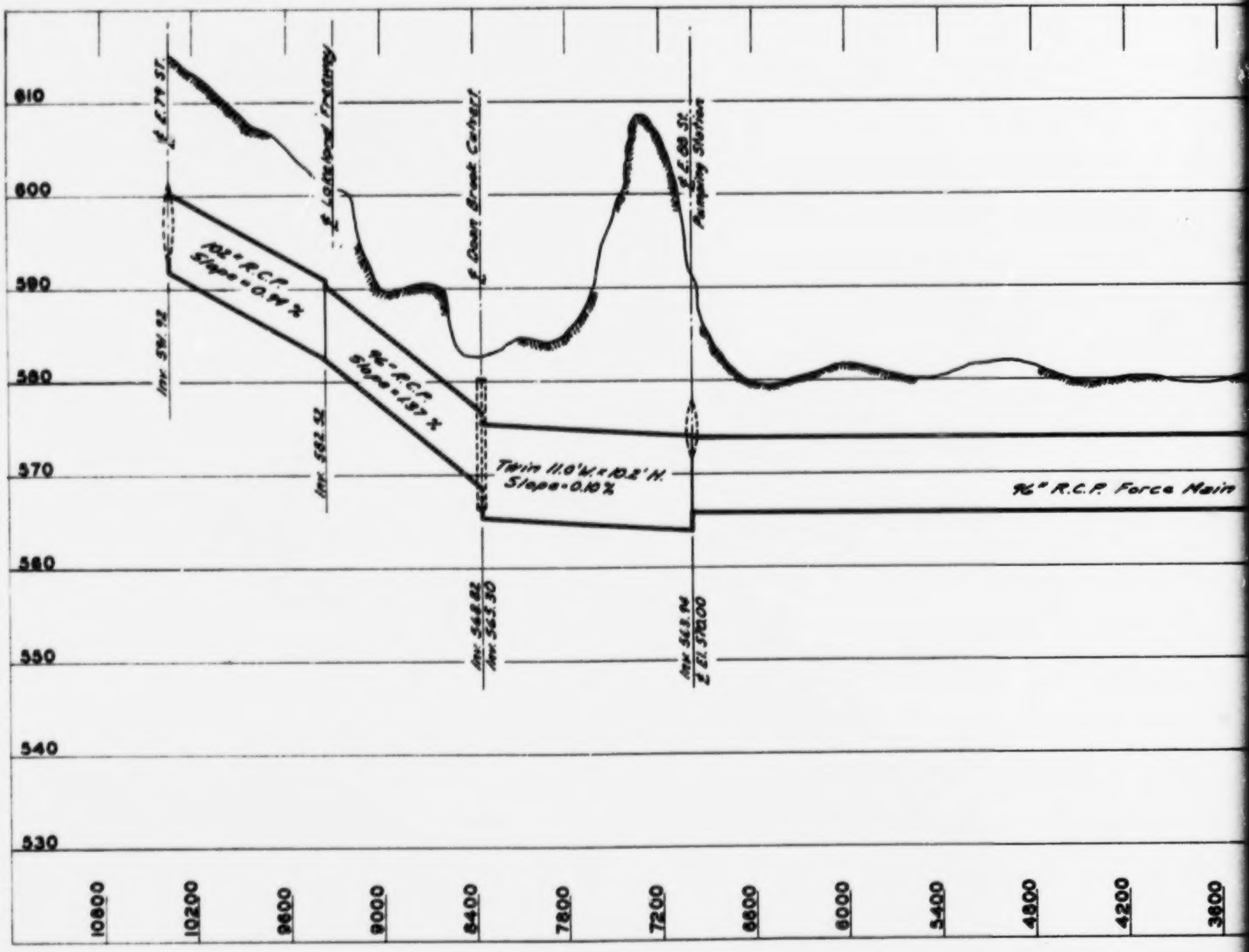

40
STEEL SHEET PILE CELL
11.2'
17.56'
72.10'
66.84'
DREDGE ACCESS
PLAN
CONCRETE OR ASPHALT CAP
10'
EL. 573.3
H= 10' TO 45'
H+10'/2
SECTION
CONSTRUCTION DETAILS
42
40
1100'
1200'
350'
750' R.
30
30
1080'
1450'
50'
400' R.
27
24
24
18
18
12
CAUSEWAY
EASTERLY OVERFLOW CONDUIT UNDER THE ROADWAY
TREATED EASTERLY EFFLUENT OUTFALL
CAUSEWAY ROAD
FOR DETAILS OF THE EAST COLLECTION SYSTEM, SEE FIG. 6-2
EASTERLY WASTEWATER TREATMENT PLANT
WHITE CITY BEACH
2000

FIGURE 5-1
FEASIBILITY STUDY OF
STABILIZATION-RETENTION BASIN
BASIN DESIGN

B


96" R.C.P. FORCE MAIN
LAKELAND
96" R.C.P.
HOLDING BASIN
PUMPING STATION
102" R.C.P.
TWIN 11.0'x10.2'
TRASH RACK
LAKE
SHORE
BLVD.
ST.
E. 72
E. 79
ST.
E. 82
ST.
DOAN BR.
E. 88
E. 99
E. 105
FREEWAY

PLAN


610
600
590
580
570
560
550
540
530
℄ E.79 ST.
℄ Lakeland Freeway
℄ Doan Brook Culvert
℄ E.88 St. Pumping Station
102" R.C.P. Slope=0.94%
96" R.C.P. Slope=1.37%
Twin 11.0'W.×10.2'H. Slope=0.10%
96" R.C.P. Force Main
Inv. 591.92
Inv. 582.52
Inv. 568.82
Inv. 565.30
Inv. 563.94
℄ El. 570.00
10800
10200
9600
9000
8400
7800
7200
6600
6000
5400
4800
4200
3600

PR

0

HAVENS AND EMERSON

A

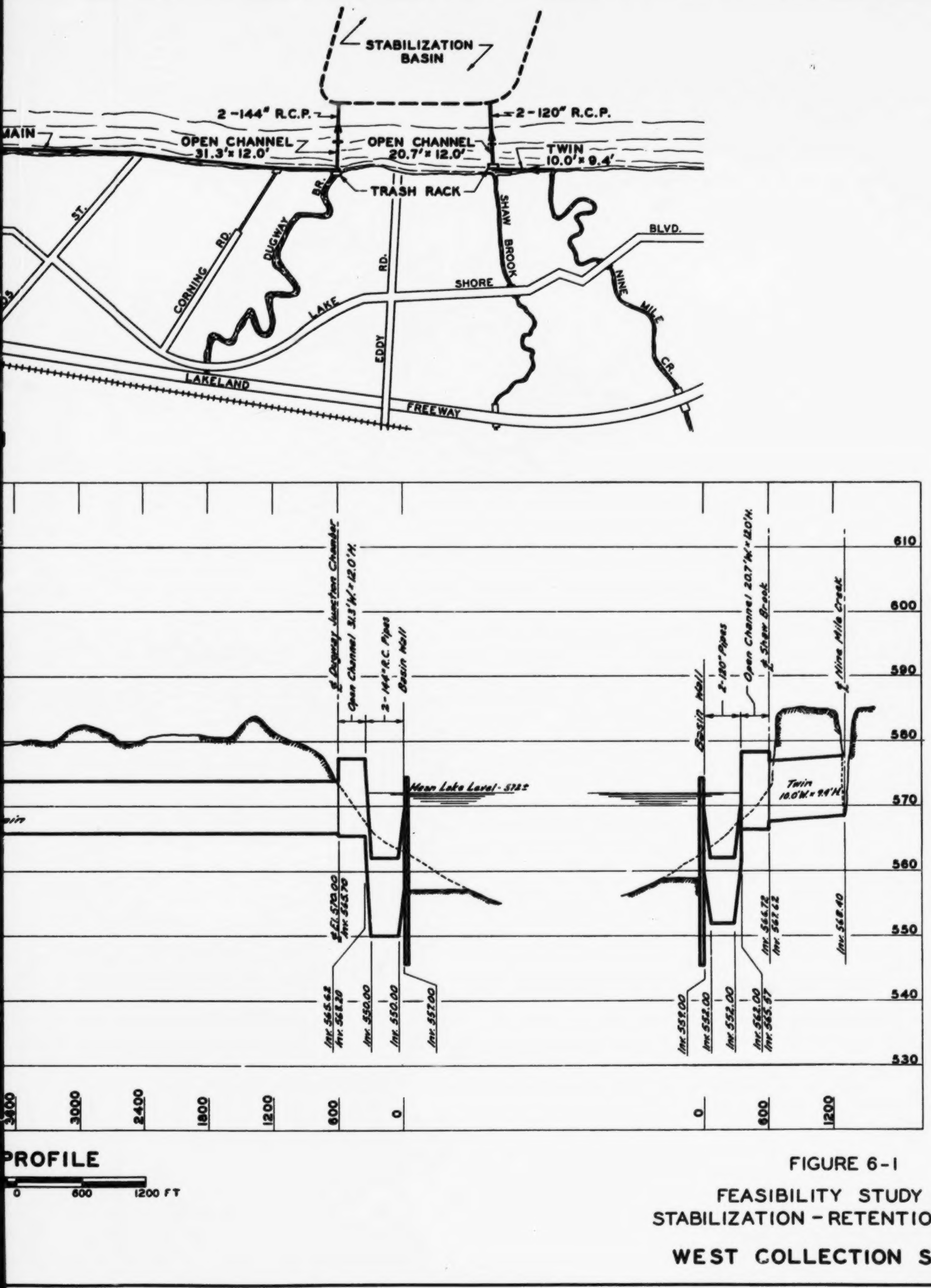
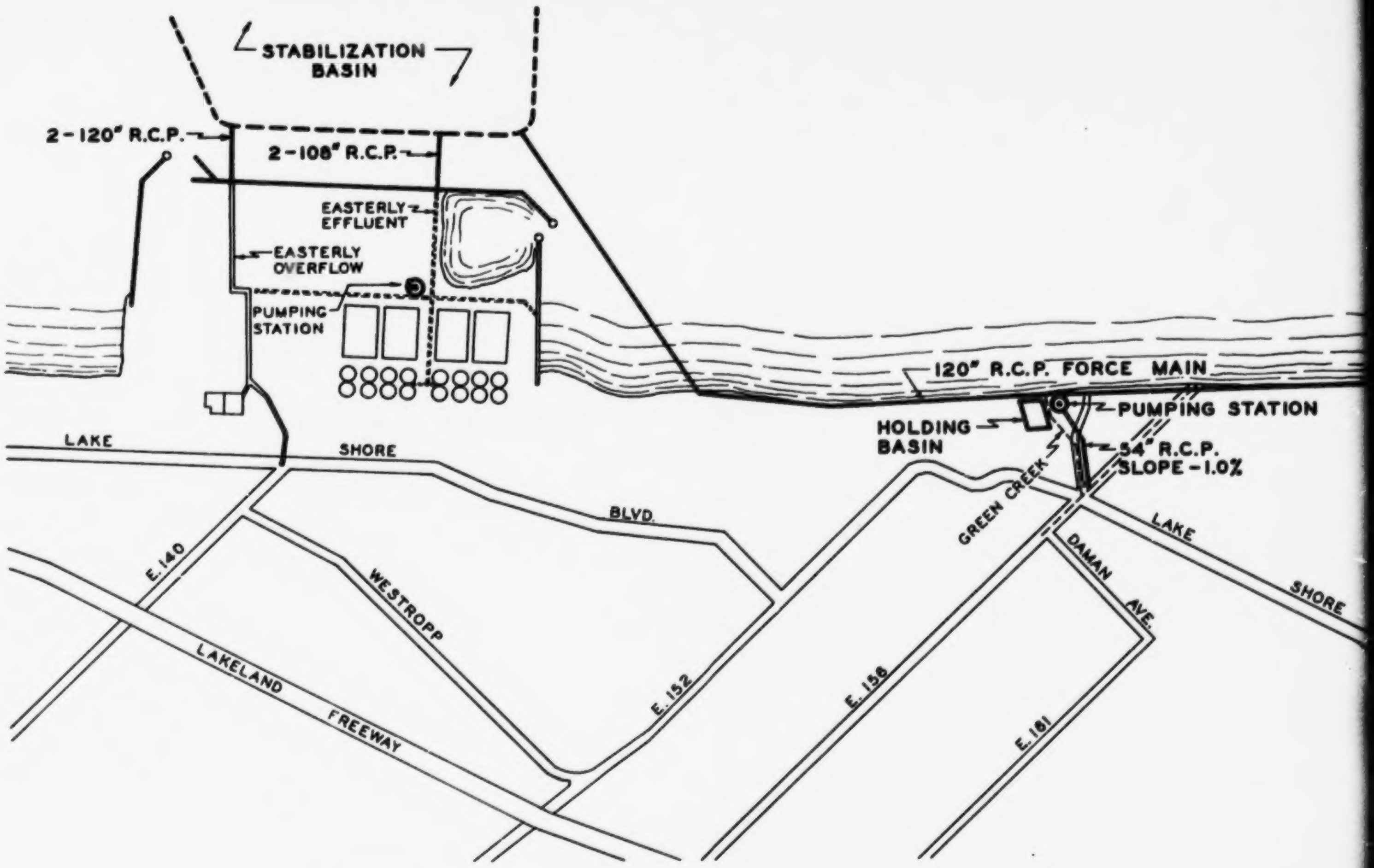

STABILIZATION BASIN
2-144" R.C.P.
2-120" R.C.P.
MAIN
OPEN CHANNEL 31.3'×12.0'
OPEN CHANNEL 20.7'×12.0'
TWIN 10.0'×9.4'
TRASH RACK
ST.
CORNING RD.
DUGWAY BR.
RD.
EDDY
SHAW BROOK
SHORE
LAKE
BLVD.
NINE MILE CR.
LAKELAND
FREEWAY
℄ Dugway Junction Chamber
Open Channel 31.3'W.×12.0'H.
2-144" R.C. Pipes
Basin Wall
Mean Lake Level - 572±
℄ El. 578.00
Inv. 565.70
Inv. 565.62
Inv. 568.20
Inv. 550.00
Inv. 550.00
Inv. 557.00
Basin Wall
2-120" Pipes
Open Channel 20.7'W.×12.0'H.
℄ Shaw Brook
℄ Nine Mile Creek
Twin 10.0'W.×9.4'H.
Inv. 566.72
Inv. 567.62
Inv. 568.40
Inv. 559.00
Inv. 552.00
Inv. 552.00
Inv. 562.00
Inv. 565.57
610
600
590
580
570
560
550
540
530
3600
3000
2400
1800
1200
600
0
0
600
1200
PROFILE
0
600
1200 FT

FIGURE 6-1

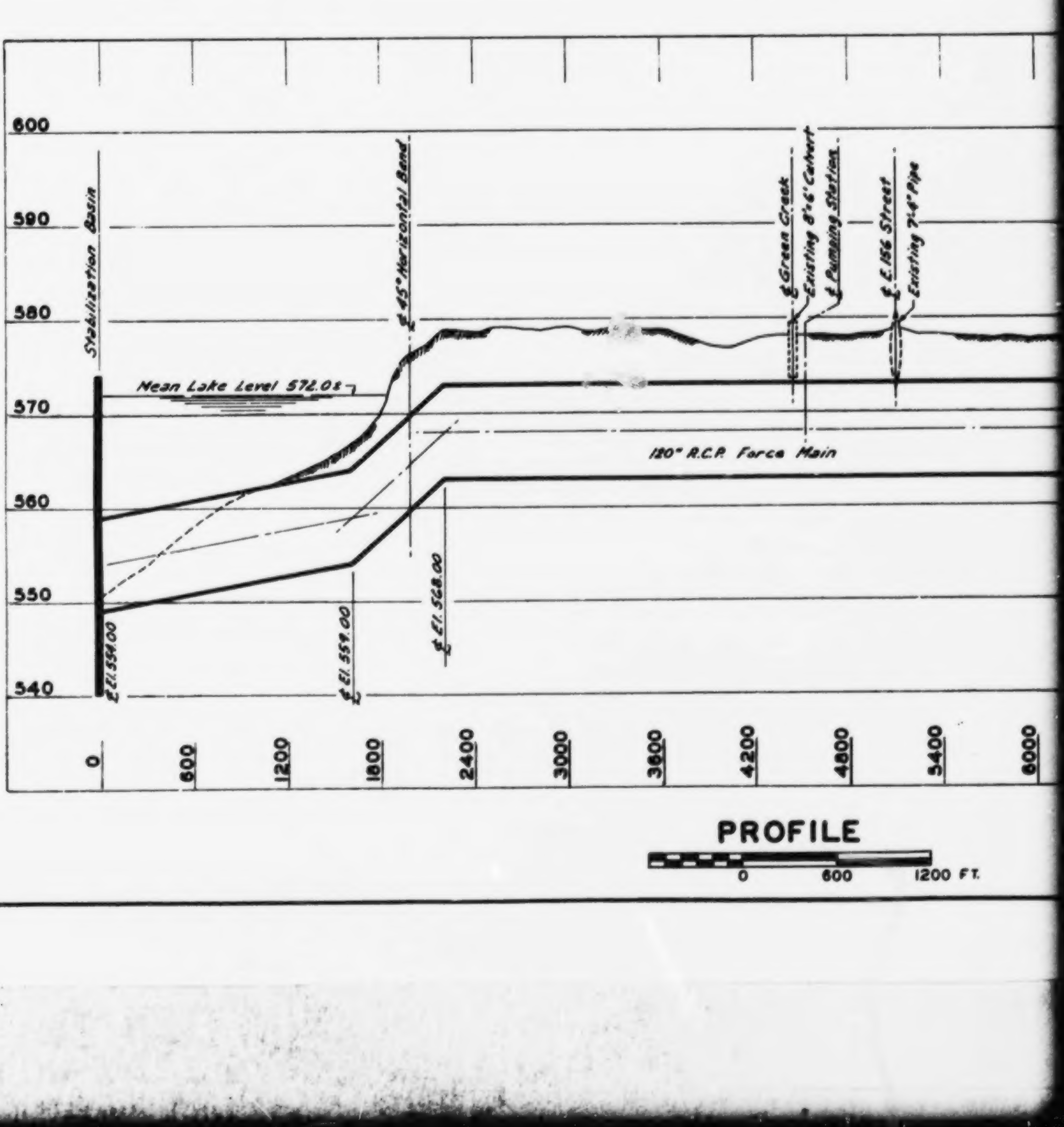
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN

WEST COLLECTION SYSTEM

B


STABILIZATION BASIN
2-120" R.C.P.
2-108" R.C.P.
EASTERLY EFFLUENT
EASTERLY OVERFLOW
PUMPING STATION
120" R.C.P. FORCE MAIN
PUMPING STATION
HOLDING BASIN
54" R.C.P. SLOPE - 1.0%
GREEN CREEK
LAKE SHORE BLVD.
LAKE SHORE
E. 140
WESTROPP
LAKELAND FREEWAY
E. 152
E. 156
DAMAN AVE.
E. 181

PLAN

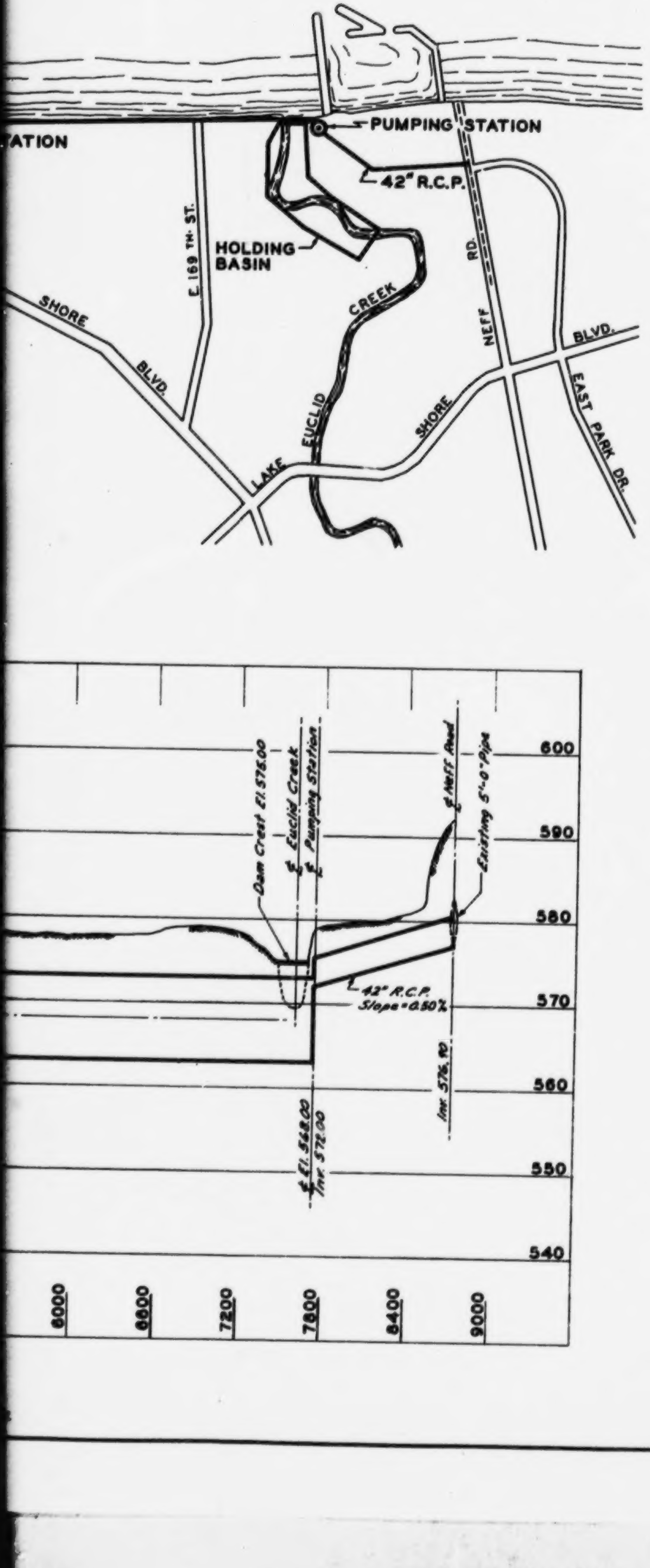

600
590
580
570
560
550
540
Stabilization Basin
℄ 45° Horizontal Bend
℄ Green Creek
Existing 8'-6' Culvert
℄ Pumping Station
℄ E. 186 Street
Existing 7'-6" Pipe
Mean Lake Level 572.0±
120" R.C.P. Force Main
℄ El. 554.00
℄ El. 559.00
℄ El. 568.00
0
600
1200
1800
2400
3000
3600
4200
4800
5400
6000

PROFILE

0 600 1200 FT.

HAVENS AND EMERSON

A



TATION
PUMPING STATION
42" R.C.P.
HOLDING BASIN
E. 169 TH. ST.
SHORE
BLVD.
CREEK
EUCLID
LAKE
NEFF
RD.
SHORE
BLVD.
EAST PARK DR.
Dam Crest El. 575.00
℄ Euclid Creek
℄ Pumping Station
℄ Neff Road
Existing 5'-0" Pipe
42" R.C.P.
Slope = 0.50%
℄ El. 568.00
Inv. 572.00
Inv. 576.90
600
590
580
570
560
550
540
6000
6600
7200
7800
8400
9000

FIGURE 6-2

FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN

EAST COLLECTION SYSTEM

B

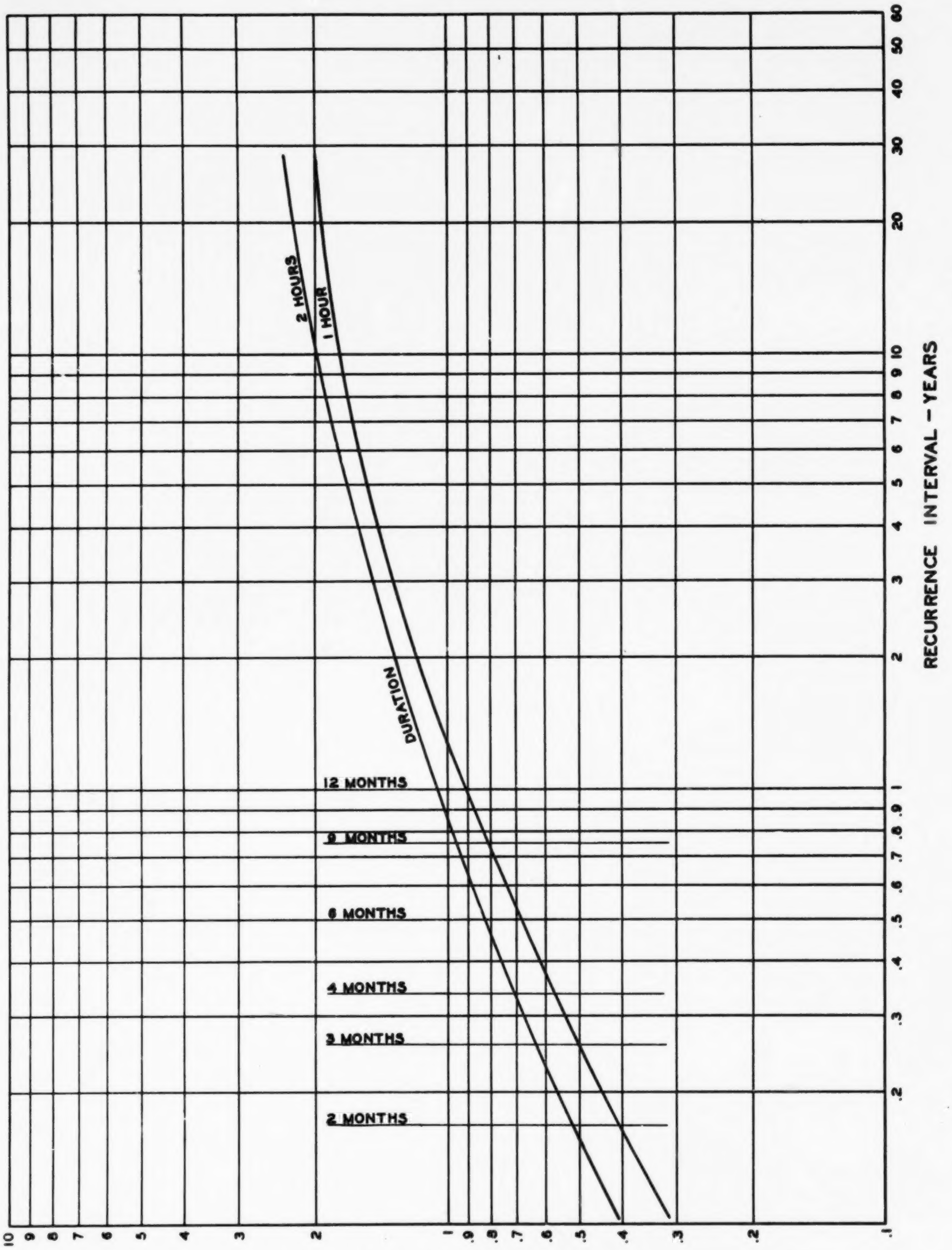


FIGURE A-1
FEASIBILITY STUDY OF
STABILIZATION-RETENTION BASIN
FREQUENCY-DEPTH CURVES

HAVENS AND EMERSON

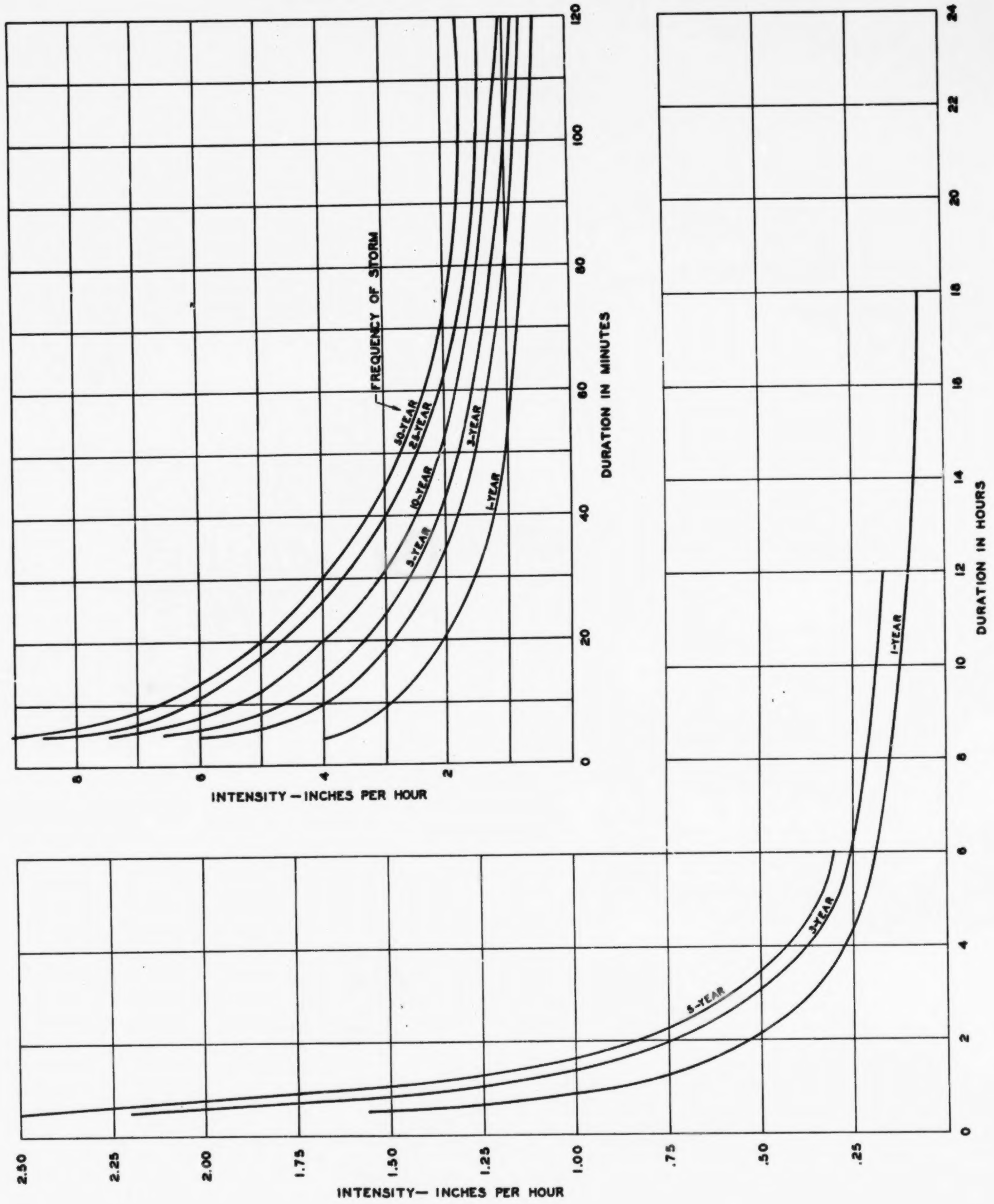


FIGURE A-2
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN

INTENSITY-DURATION CURVES

HAVENS AND EMERSON

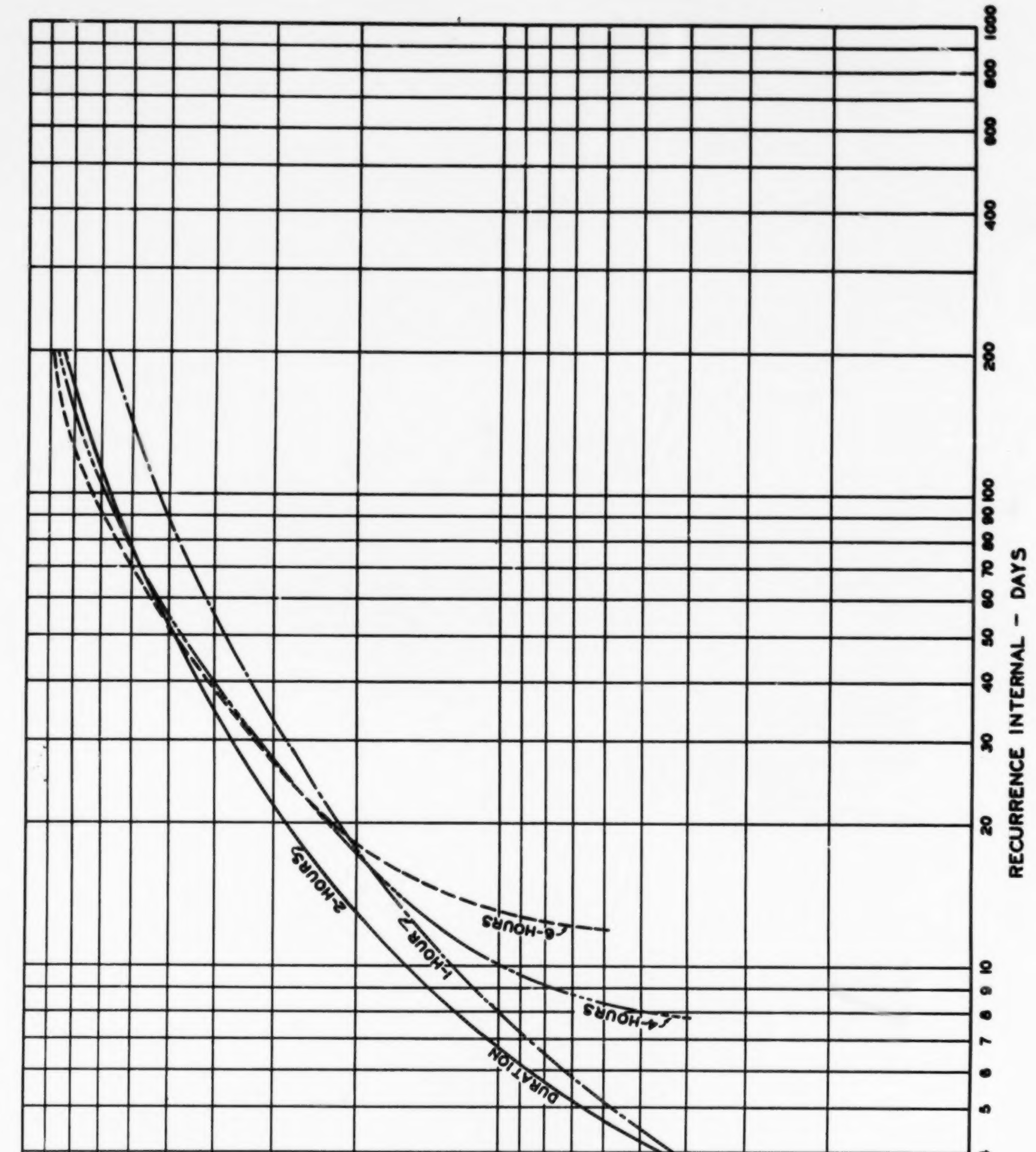
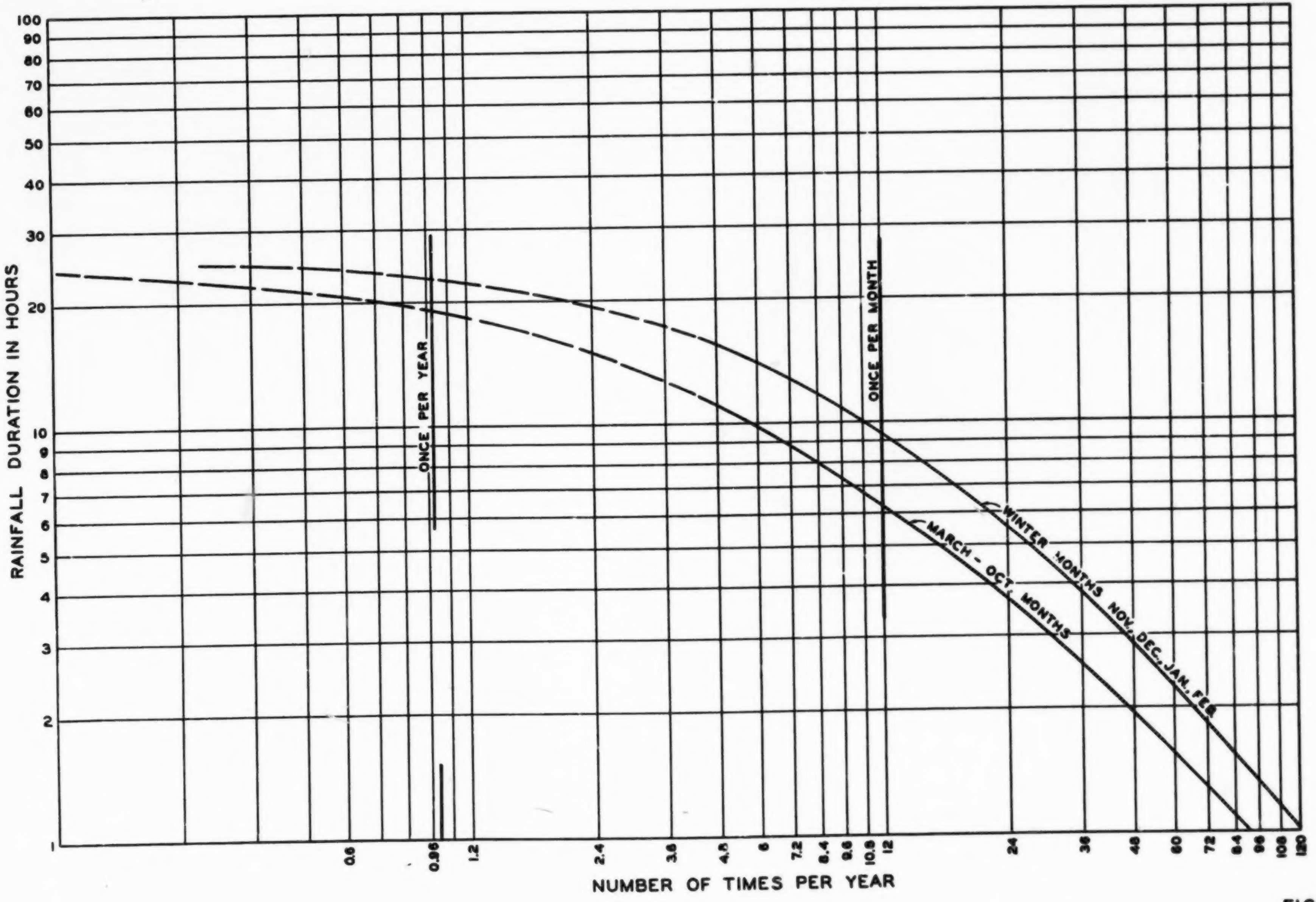


FIGURE A-3
FEASIBILITY STUDY OF
STABILIZATION-RETENTION BASIN

HAVENS AND EMERSON

FREQUENCY-DEPTH CURVES



HAVENS AND EMERSON

FIGURE A-4
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN

LENGTH OF STORMS

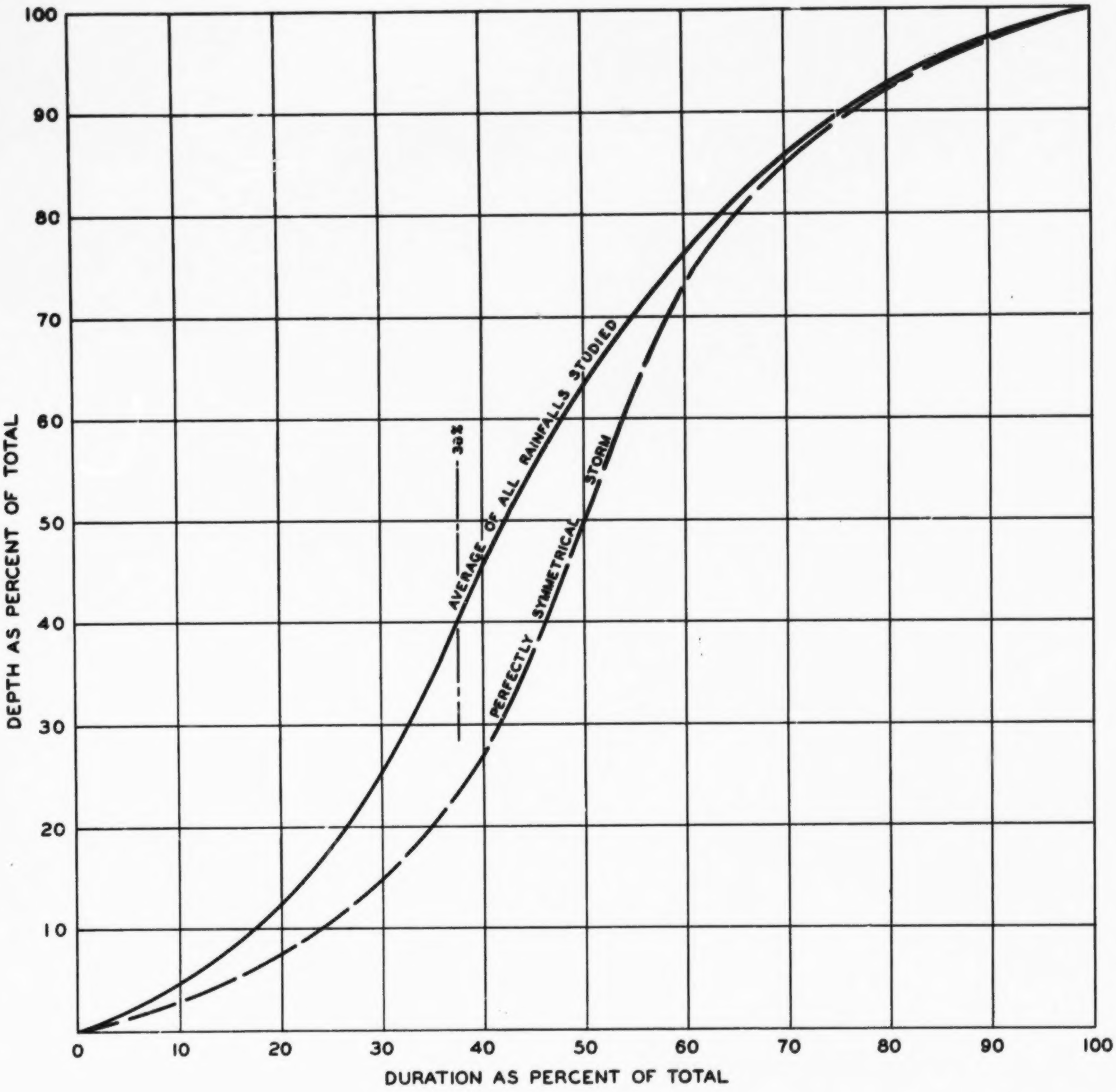
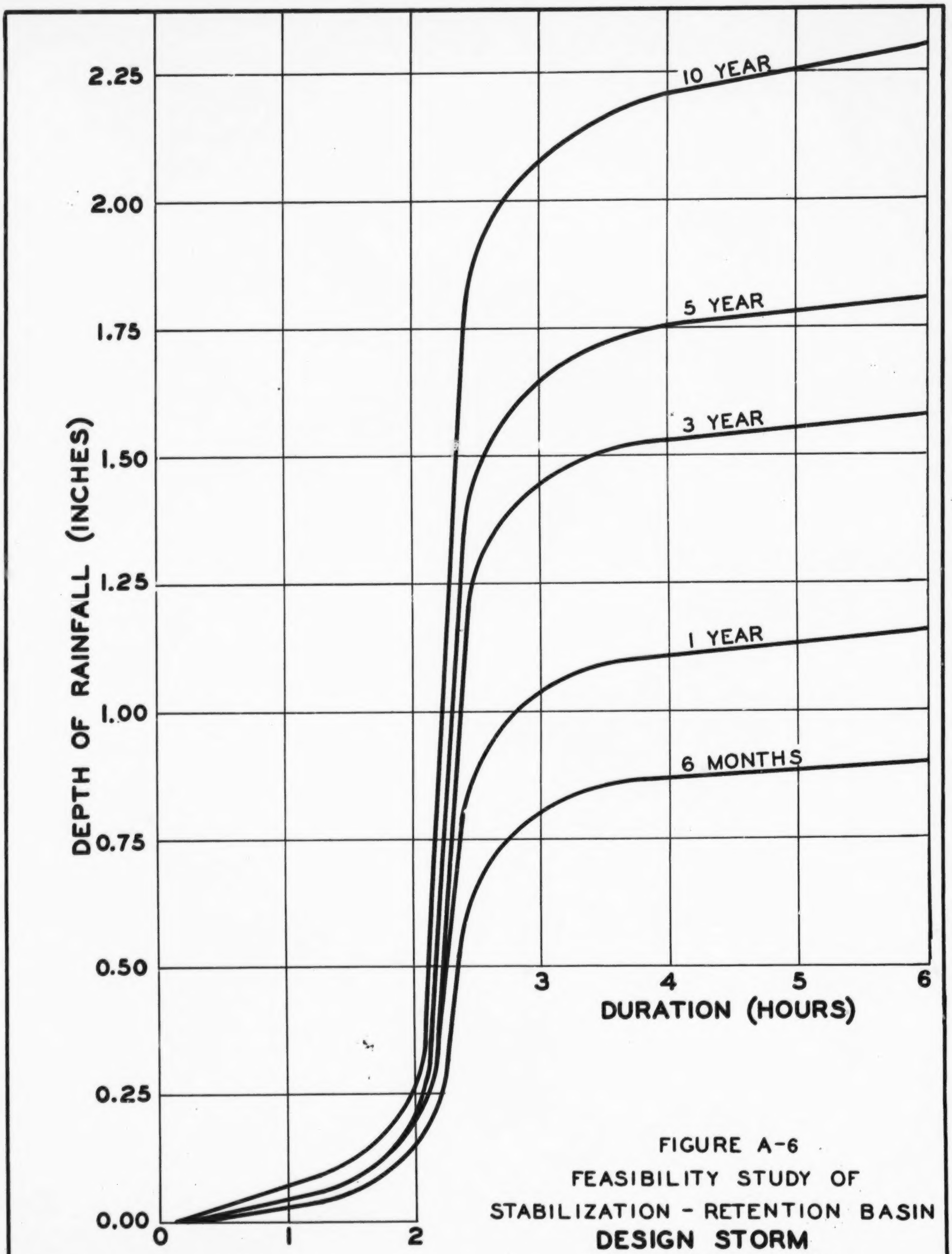


FIGURE A-5
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN

**AVERAGE MASS DIAGRAM
OF RAINFALLS STUDIED**

HAVENS AND EMERSON



HAVENS AND EMERSON

FIGURE A-6
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN
DESIGN STORM
MASS DIAGRAMS

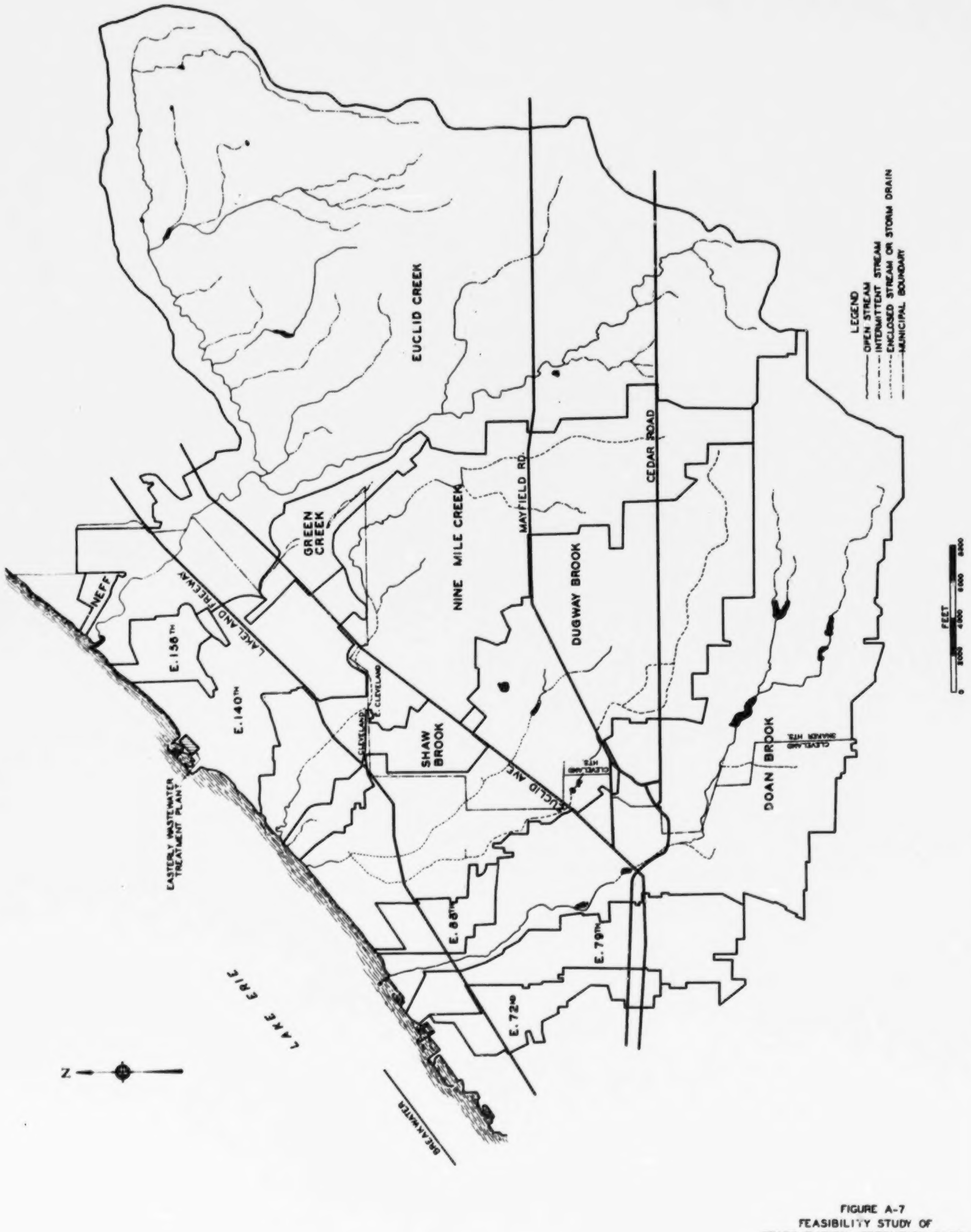
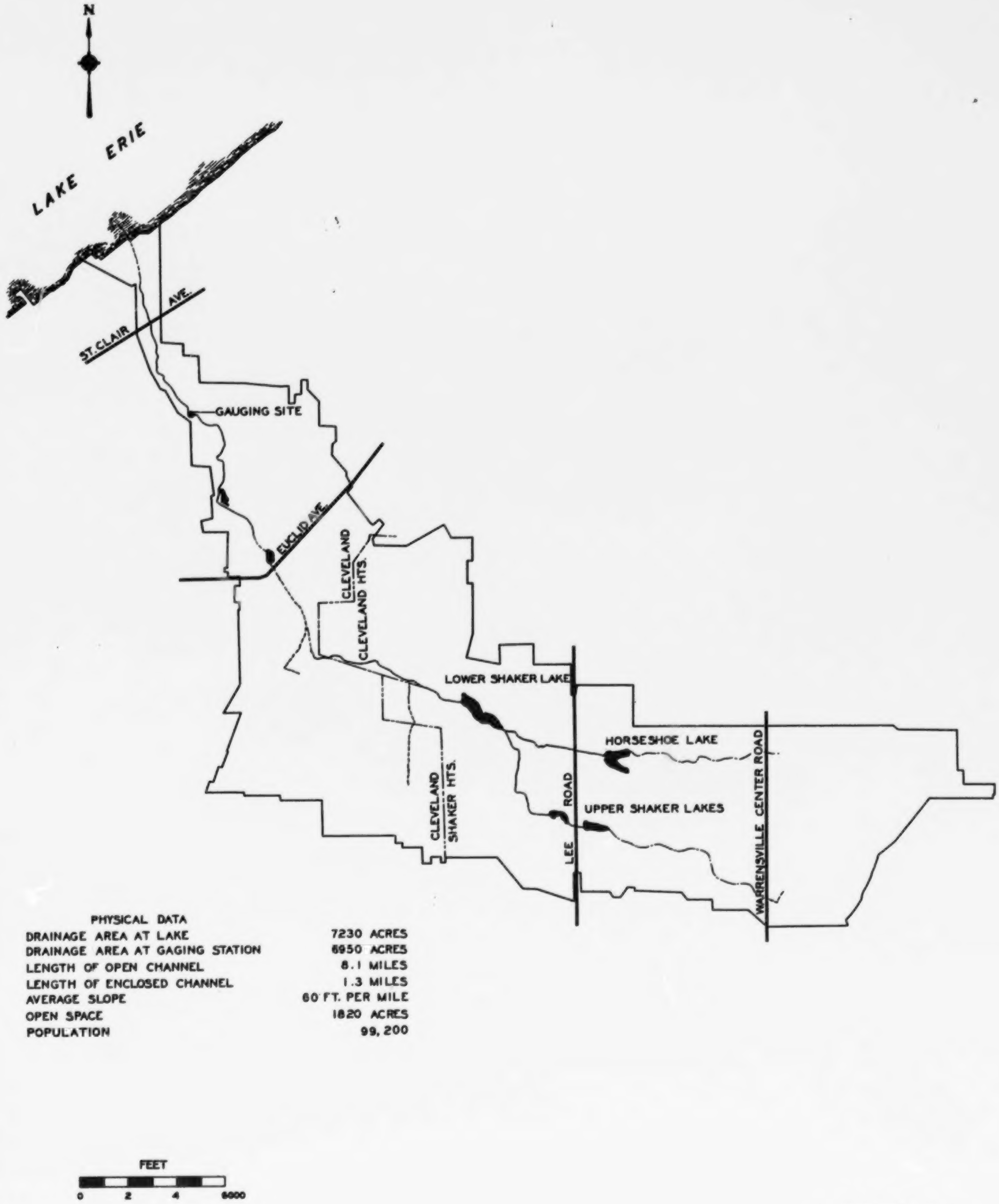

EUCLID CREEK
GREEN CREEK
NINE MILE CREEK
DUGWAY BROOK
SHAW BROOK
DOAN BROOK
NEFF
E. 156TH
E. 140TH
E. 88TH
E. 79TH
E. 72ND
MAYFIELD RD
CEDAR ROAD
EUCLID AVE
LAKELAND FREEWAY
CLEVELAND
E. CLEVELAND
CLEVELAND HTS
SHAKER HTS
EASTERLY WASTEWATER TREATMENT PLANT
LAKE ERIE
BREAKWATER
N
LEGEND
OPEN STREAM
INTERMITTENT STREAM
ENCLOSED STREAM OR STORM DRAIN
MUNICIPAL BOUNDARY
FEET
0
2000
4000
6000
8000

FIGURE A-7
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN

STUDY AREA

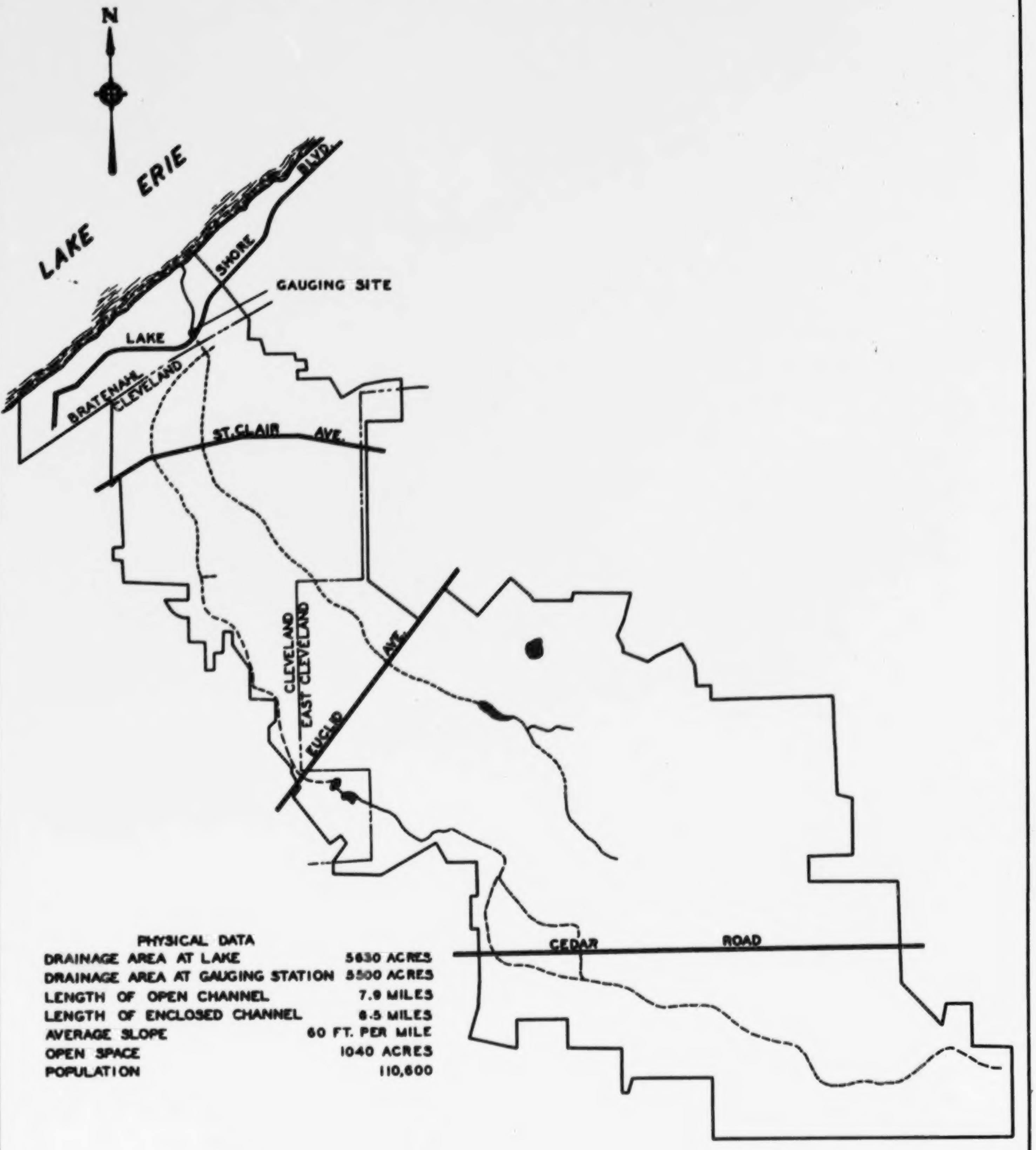
HAVENS AND EMERSON


N
LAKE ERIE
ST. CLAIR AVE.
GAUGING SITE
EUCLID AVE.
CLEVELAND
CLEVELAND HTS.
LOWER SHAKER LAKE
HORSESHOE LAKE
CLEVELAND SHAKER HTS.
LEE ROAD
UPPER SHAKER LAKES
WARRENSVILLE CENTER ROAD
FEET
0
2
4
6000

PHYSICAL DATA	
DRAINAGE AREA AT LAKE	7230 ACRES
DRAINAGE AREA AT GAGING STATION	6950 ACRES
LENGTH OF OPEN CHANNEL	8.1 MILES
LENGTH OF ENCLOSED CHANNEL	1.3 MILES
AVERAGE SLOPE	60 FT. PER MILE
OPEN SPACE	1820 ACRES
POPULATION	99, 200

FIGURE A-8
FEASIBILITY STUDY OF
STABILIZATION – RETENTION BASIN
DOAN BROOK

HAVENS AND EMERSON

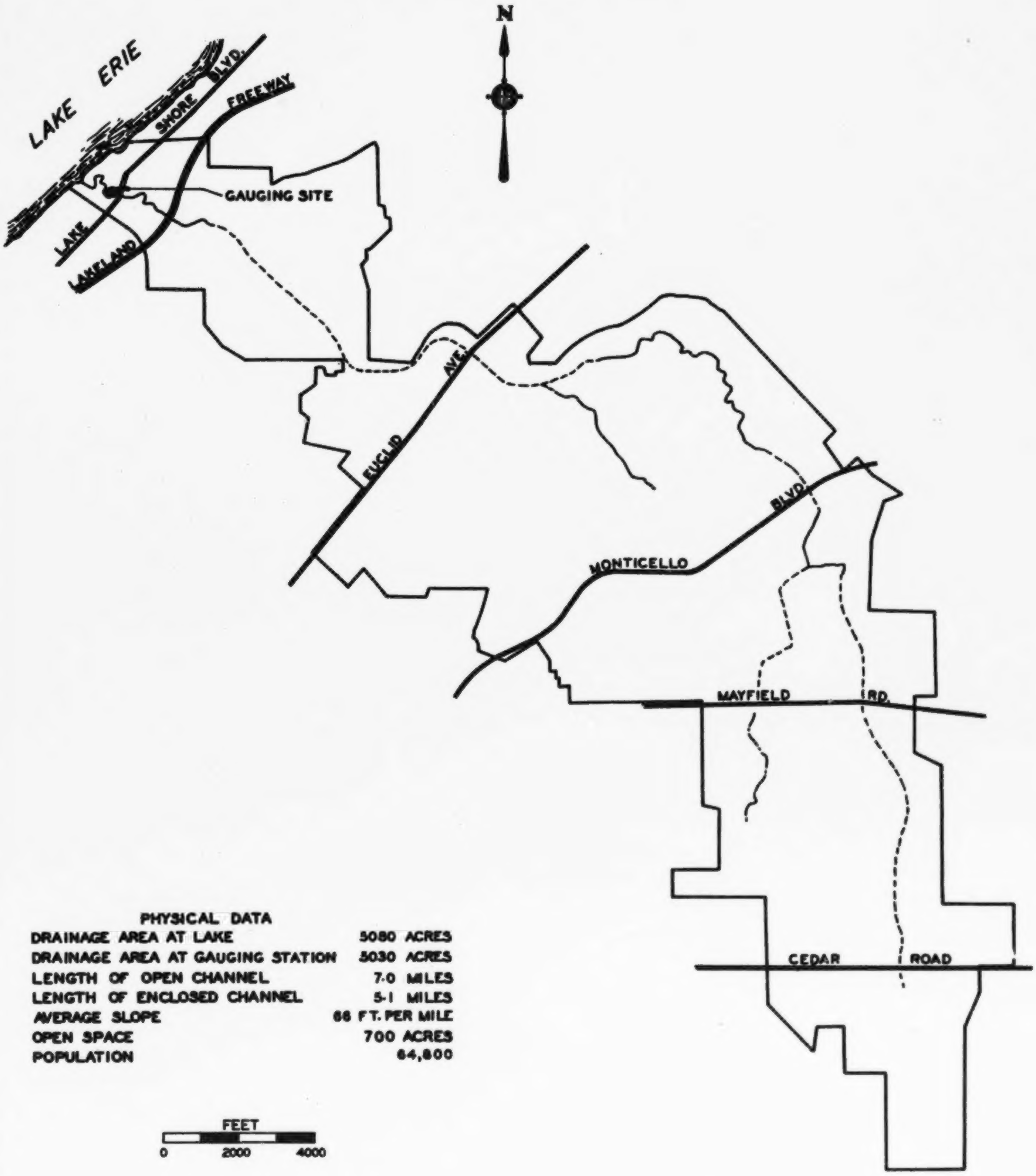

N
LAKE ERIE
SHORE BLVD.
GAUGING SITE
LAKE
BRATENAHL
CLEVELAND
ST. CLAIR AVE.
CLEVELAND
EAST CLEVELAND
EUCLID AVE.
CEDAR ROAD
FEET
0
2000
4000

PHYSICAL DATA	
DRAINAGE AREA AT LAKE	5630 ACRES
DRAINAGE AREA AT GAUGING STATION	5500 ACRES
LENGTH OF OPEN CHANNEL	7.9 MILES
LENGTH OF ENCLOSED CHANNEL	8.5 MILES
AVERAGE SLOPE	60 FT. PER MILE
OPEN SPACE	1040 ACRES
POPULATION	110,600

FIGURE A-9
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN

DUGWAY BROOK

HAVENS AND EMERSON

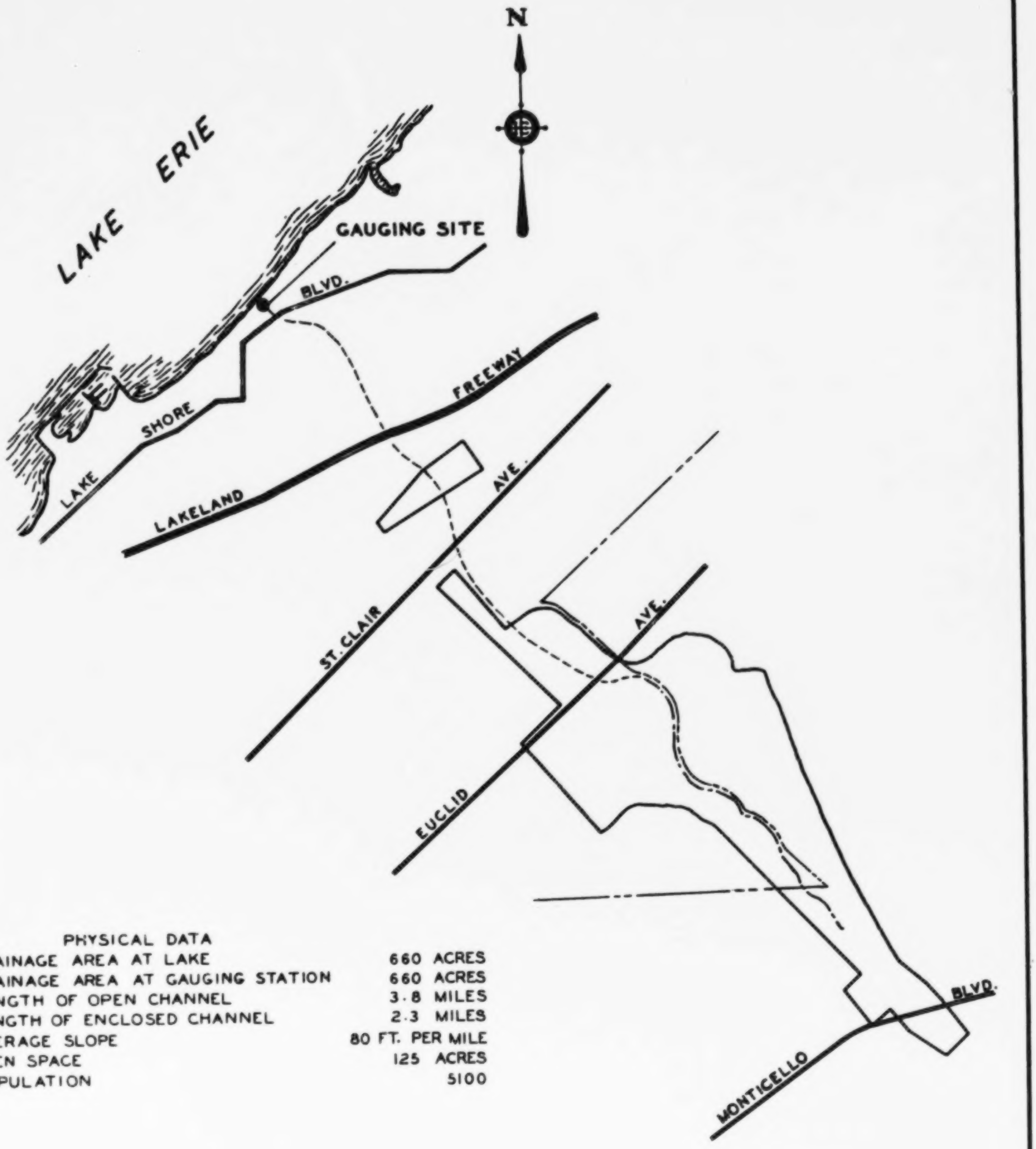

N
LAKE ERIE
SHORE BLVD.
FREEWAY
GAUGING SITE
LAKE
LAKELAND
EUCLID AVE.
MONTICELLO BLVD.
MAYFIELD RD.
CEDAR ROAD
FEET
0
2000
4000

PHYSICAL DATA	
DRAINAGE AREA AT LAKE	5080 ACRES
DRAINAGE AREA AT GAUGING STATION	5030 ACRES
LENGTH OF OPEN CHANNEL	7.0 MILES
LENGTH OF ENCLOSED CHANNEL	5.1 MILES
AVERAGE SLOPE	66 FT. PER MILE
OPEN SPACE	700 ACRES
POPULATION	64,800

FIGURE A-10
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN

NINE MILE CREEK

HAVENS AND EMERSON

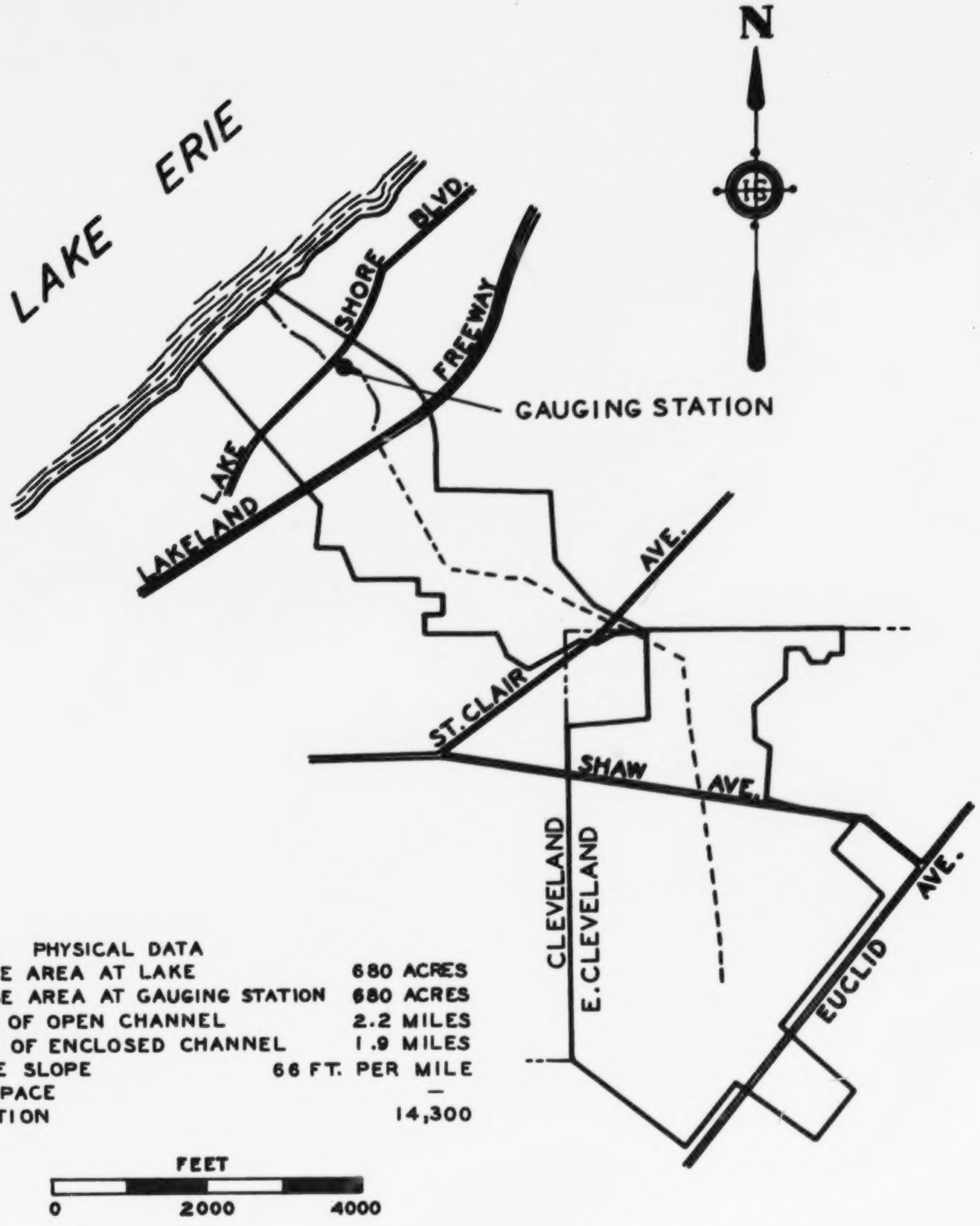

N
LAKE ERIE
GAUGING SITE
LAKE SHORE BLVD.
LAKELAND FREEWAY
ST. CLAIR AVE.
EUCLID AVE.
MONTICELLO BLVD.
FEET
0
2000
4000

PHYSICAL DATA	
DRAINAGE AREA AT LAKE	660 ACRES
DRAINAGE AREA AT GAUGING STATION	660 ACRES
LENGTH OF OPEN CHANNEL	3.8 MILES
LENGTH OF ENCLOSED CHANNEL	2.3 MILES
AVERAGE SLOPE	80 FT. PER MILE
OPEN SPACE	125 ACRES
POPULATION	5100

FIGURE A–11
FEASIBILITY STUDY OF
STABILIZATION- RETENTION BASIN

GREEN CREEK

HAVENS AND EMERSON

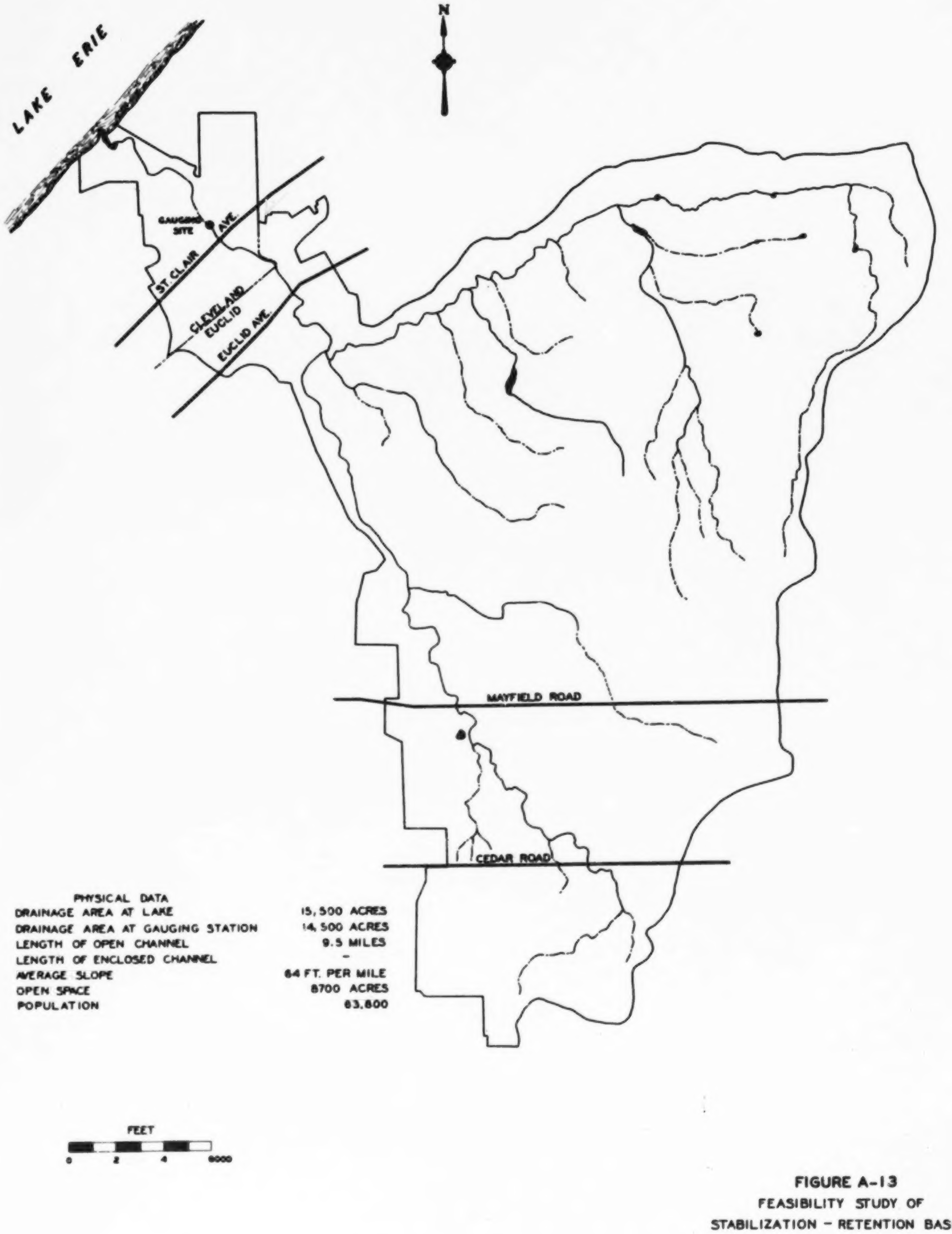

N
LAKE ERIE
SHORE BLVD.
LAKE
LAKELAND
FREEWAY
GAUGING STATION
ST. CLAIR AVE.
SHAW AVE.
CLEVELAND
E. CLEVELAND
EUCLID AVE.
FEET
0
2000
4000

PHYSICAL DATA

DRAINAGE AREA AT LAKE	680 ACRES
DRAINAGE AREA AT GAUGING STATION	680 ACRES
LENGTH OF OPEN CHANNEL	2.2 MILES
LENGTH OF ENCLOSED CHANNEL	1.9 MILES
AVERAGE SLOPE	66 FT. PER MILE
OPEN SPACE	—
POPULATION	14,300

FIGURE A-12
FEASIBILITY STUDY OF
STABILIZATION- RETENTION BASIN
SHAW BROOK

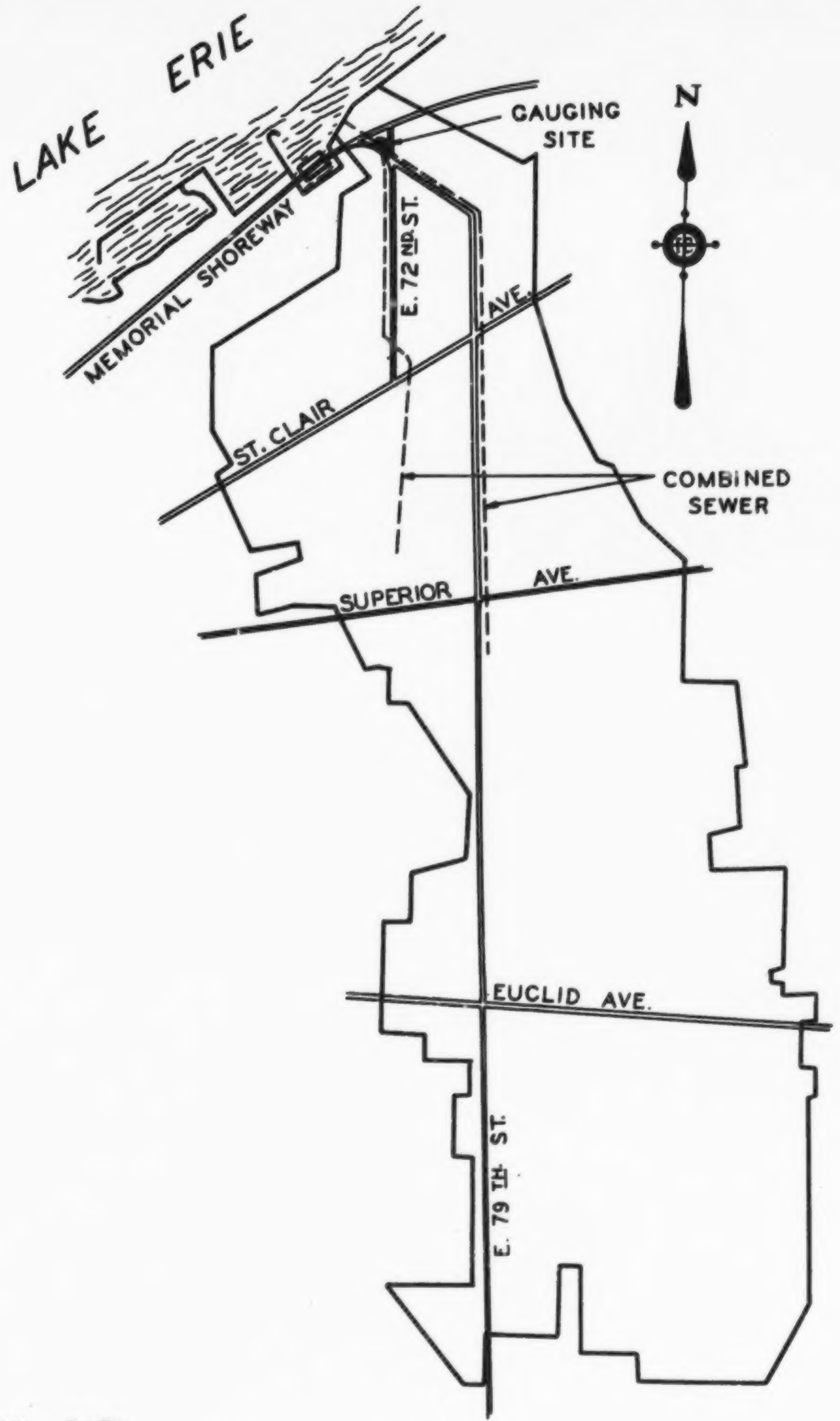
HAVENS AND EMERSON


N
LAKE ERIE
GAUGING SITE
ST. CLAIR AVE.
CLEVELAND
EUCLID
EUCLID AVE.
MAYFIELD ROAD
CEDAR ROAD
FEET
0
2
4
6000

PHYSICAL DATA	
DRAINAGE AREA AT LAKE	15,500 ACRES
DRAINAGE AREA AT GAUGING STATION	14,500 ACRES
LENGTH OF OPEN CHANNEL	9.5 MILES
LENGTH OF ENCLOSED CHANNEL	-
AVERAGE SLOPE	64 FT. PER MILE
OPEN SPACE	8700 ACRES
POPULATION	83,800

FIGURE A-13
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN
EUCLID CREEK

HAVENS AND EMERSON

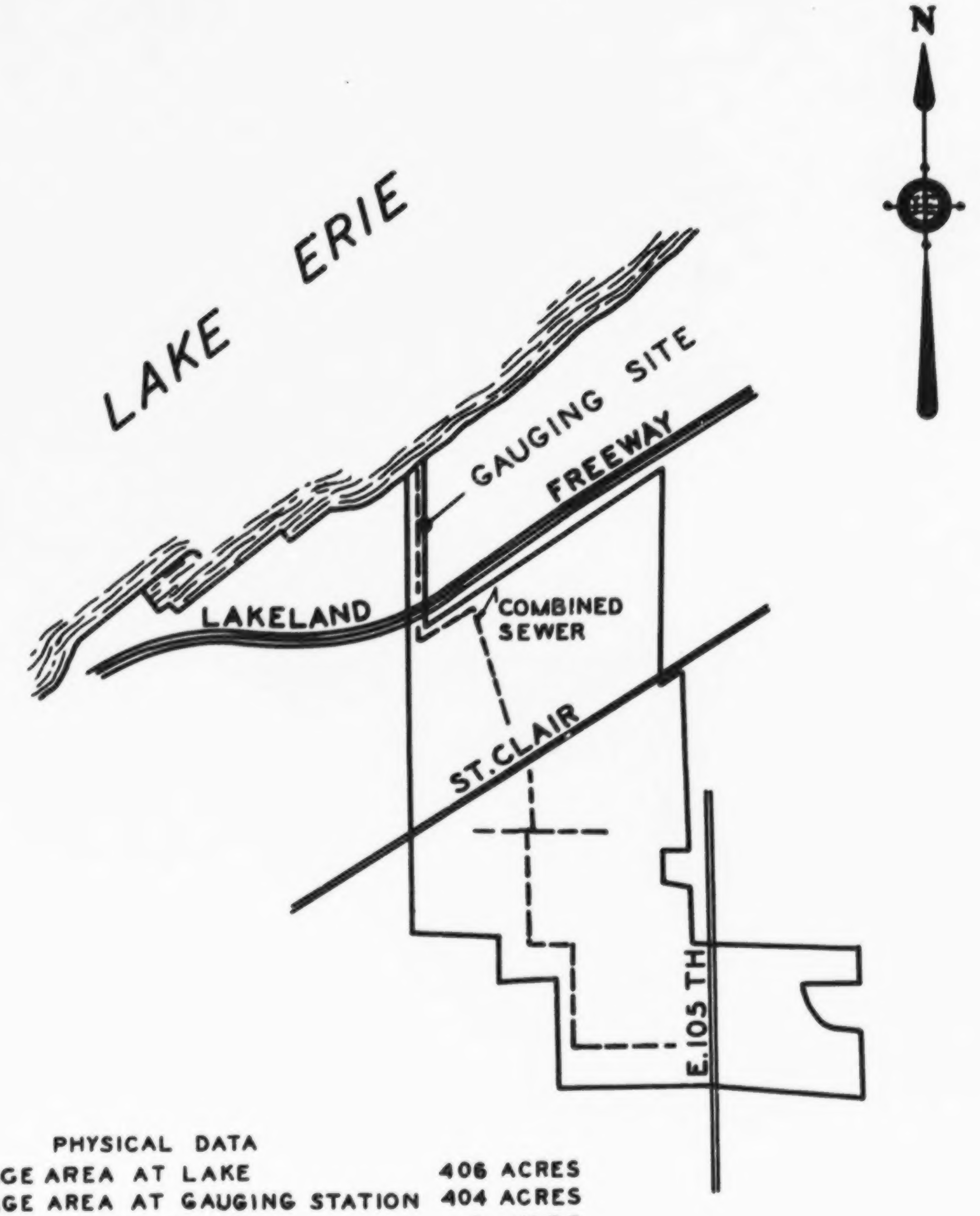

LAKE ERIE
GAUGING SITE
N
MEMORIAL SHOREWAY
E. 72ND ST.
ST. CLAIR AVE.
COMBINED SEWER
SUPERIOR AVE.
EUCLID AVE.
E. 79TH ST.

PHYSICAL DATA	
DRAINAGE AREA AT LAKE	1510 ACRES
DRAINAGE AREA AT GAUGING STATION	1490 ACRES
LENGTH OF OPEN CHANNEL	0 MILES
LENGTH OF ENCLOSED CHANNEL	4.4 MILES
AVERAGE SLOPE	——
OPEN SPACE	——
POPULATION	50,300

FIGURE A-14
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN
E. 79TH. ST. OVERFLOW

FEET
0 2 4000

HAVENS AND EMERSON


LAKE ERIE
GAUGING SITE
FREEWAY
LAKELAND
COMBINED SEWER
ST. CLAIR
E. 105 TH
N

PHYSICAL DATA

DRAINAGE AREA AT LAKE	406 ACRES
DRAINAGE AREA AT GAUGING STATION	404 ACRES
LENGTH OF OPEN CHANNEL	0 MILES
LENGTH OF ENCLOSED CHANNEL	1.6 MILES
AVERAGE SLOPE	—
OPEN SPACE	—
POPULATION	17,100

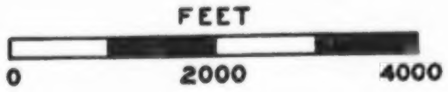
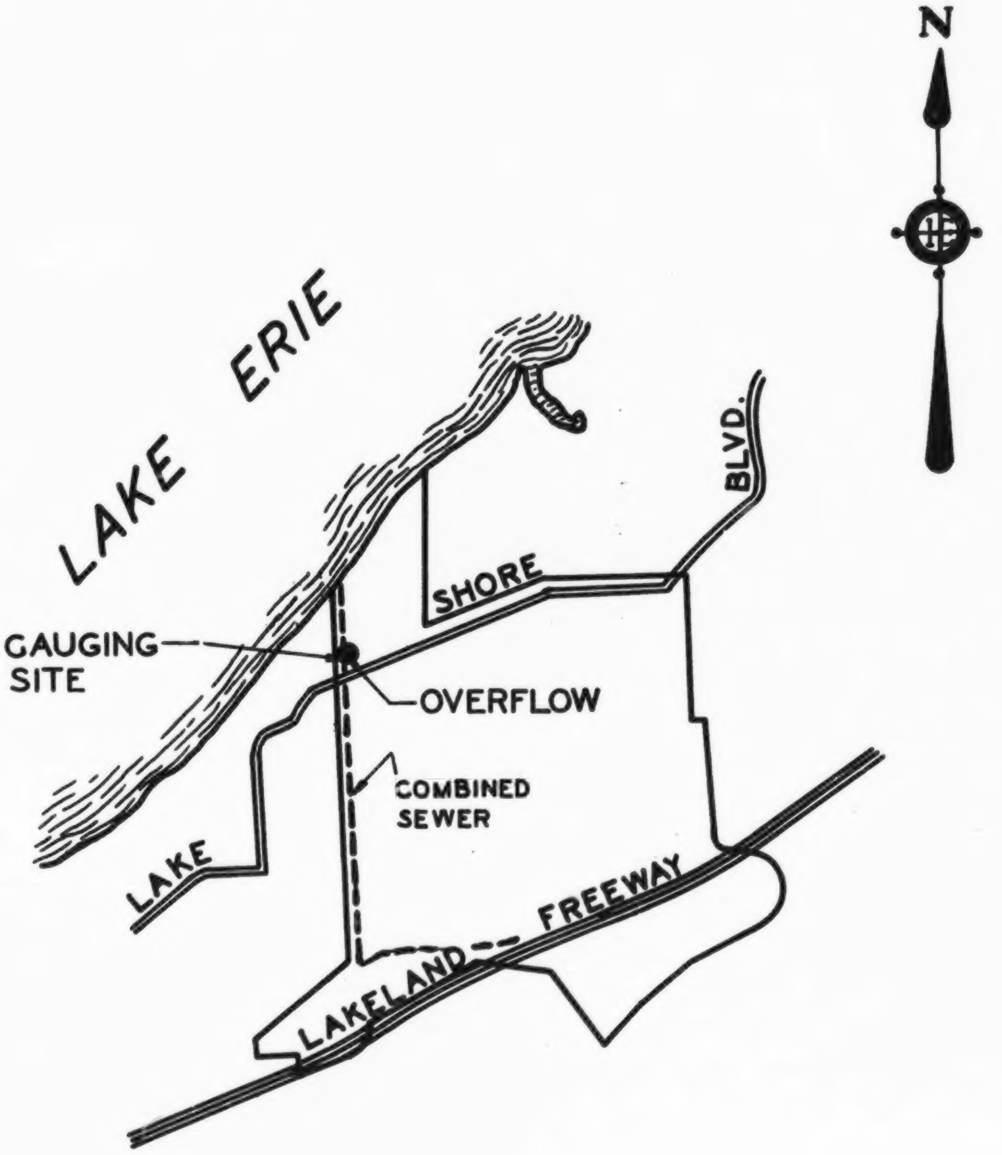

FEET
0
2000
4000

FIGURE A-15
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN
E. 88 TH. ST. OVERFLOW

HAVENS AND EMERSON


N
LAKE ERIE
SHORE BLVD.
GAUGING SITE
OVERFLOW
COMBINED SEWER
LAKE
LAKELAND FREEWAY

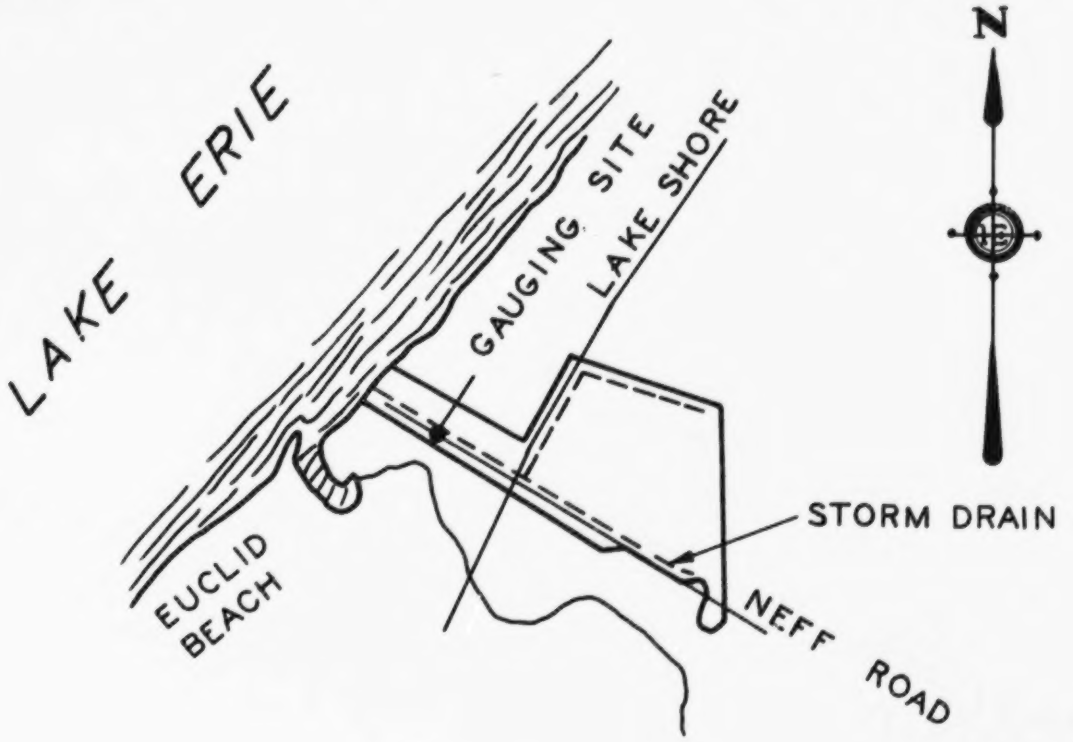
PHYSICAL DATA	
DRAINAGE AREA AT LAKE	380 ACRES
DRAINAGE AREA AT GAUGING STATION	380 ACRES
LENGTH OF OPEN CHANNEL	0 MILES
LENGTH OF ENCLOSED CHANNEL	1.8 MILES
AVERAGE SLOPE	
OPEN SPACE	80
POPULATION	5900

FEET
0 2000 4000

FIGURE A-16
FEASIBILITY STUDY OF
STABILIZATION-RETENTION BASIN

E. 156 ST. SEWER OVERFLOW

HAVENS AND EMERSON


LAKE ERIE
GAUGING SITE
LAKE SHORE
N
STORM DRAIN
EUCLID BEACH
NEFF ROAD

PHYSICAL DATA

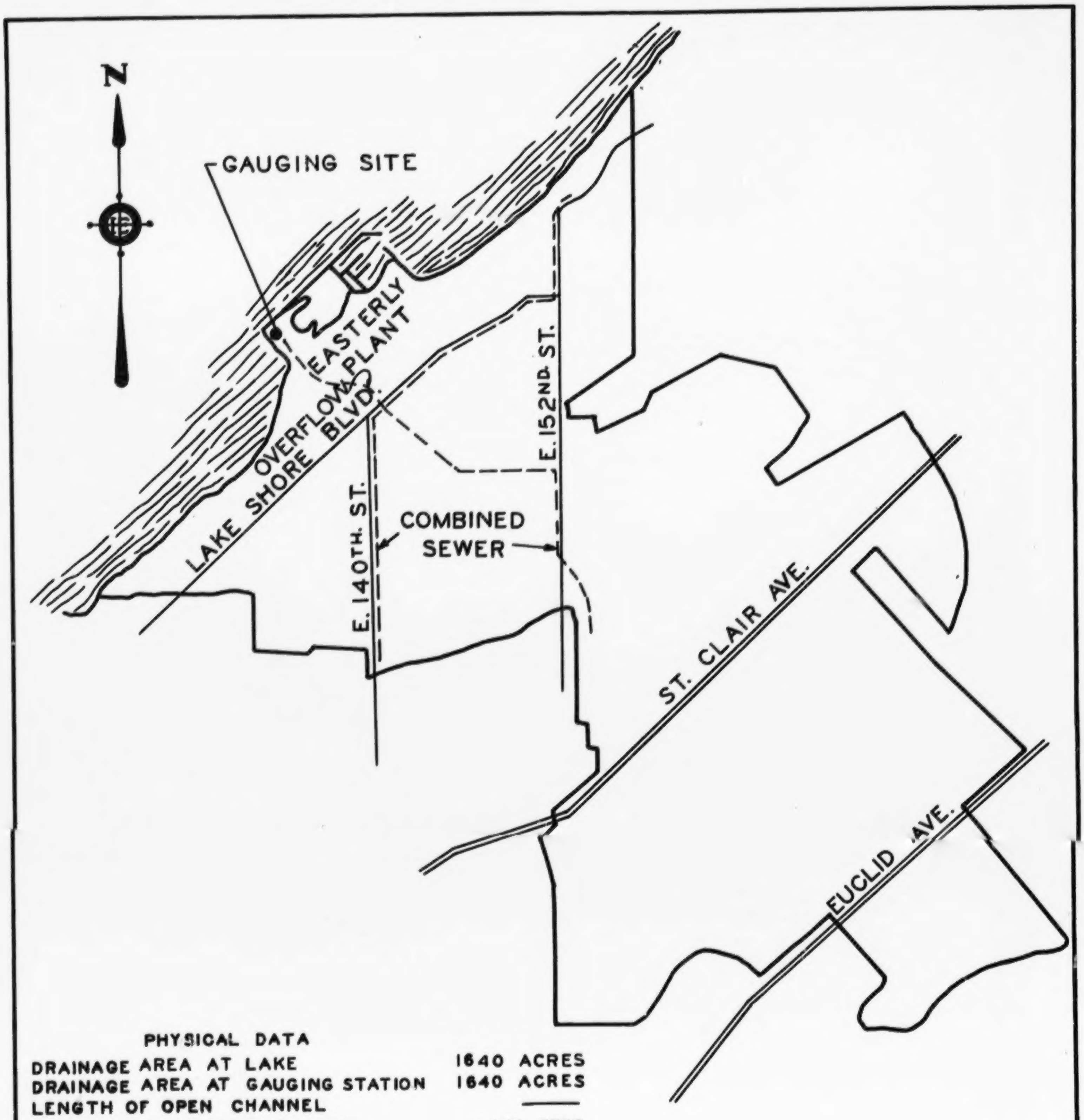
DRAINAGE AREA AT LAKE	87 ACRES
DRAINAGE AREA AT GAUGING STATION	87 ACRES
LENGTH OF OPEN CHANNEL	———
LENGTH OF ENCLOSED CHANNEL	4200 FEET
AVERAGE SLOPE	———
OPEN SPACE	———
POPULATION	1400

FEET
0
2000
4000

FIGURE A-17
FEASIBILITY STUDY OF
STABILIZATION — RETENTION BASIN

NEFF ROAD STORM DRAIN

HAVENS AND EMERSON


N
GAUGING SITE
EASTERLY PLANT
OVERFLOW
LAKE SHORE BLVD.
E. 152ND. ST.
E. 140TH. ST.
COMBINED SEWER
ST. CLAIR AVE.
EUCLID AVE.
FEET
0
2000
4000

PHYSICAL DATA

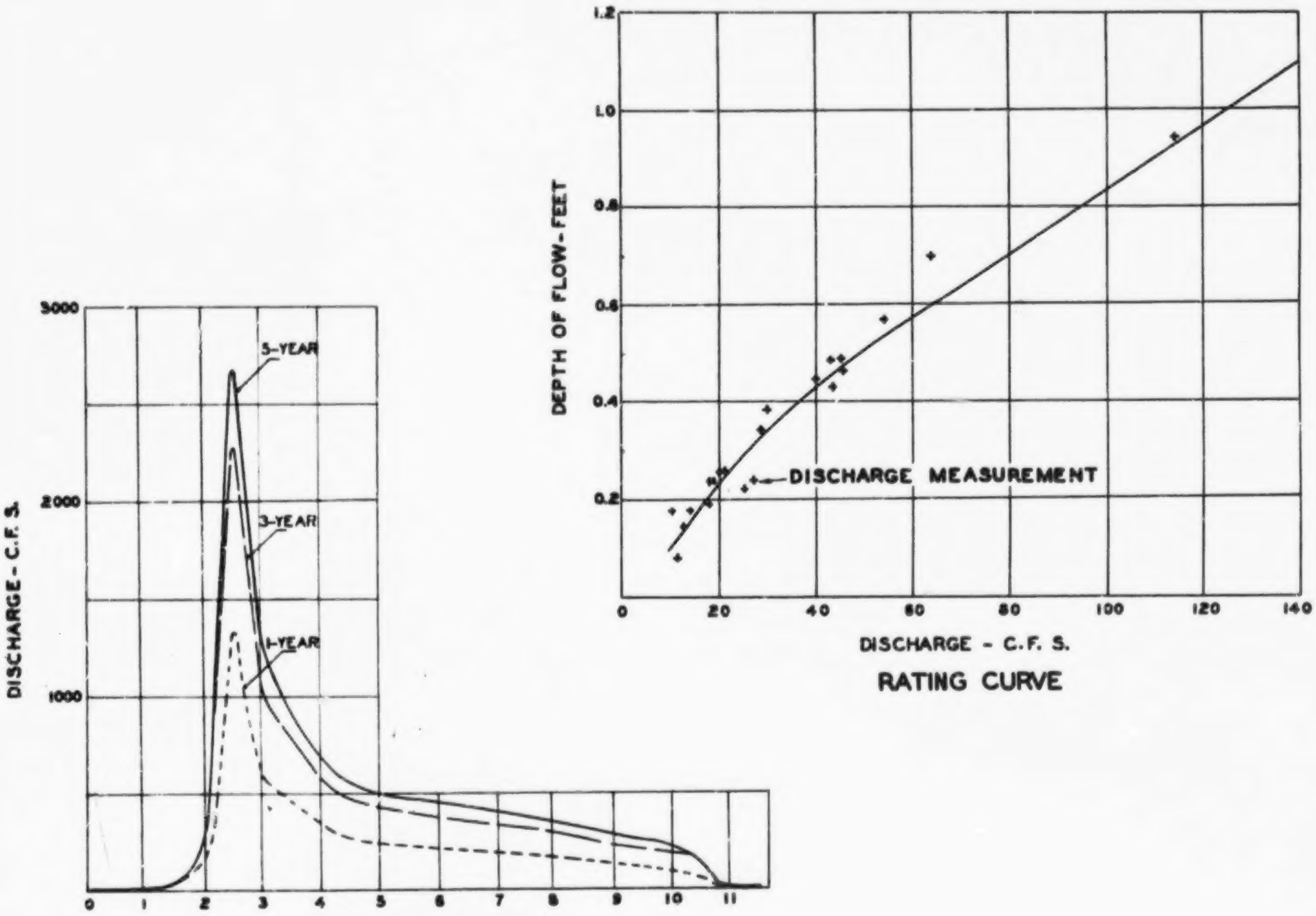
DRAINAGE AREA AT LAKE	1640 ACRES
DRAINAGE AREA AT GAUGING STATION	1640 ACRES
LENGTH OF OPEN CHANNEL	—
LENGTH OF ENCLOSED CHANNEL	14400 FEET
AVERAGE SLOPE	—
OPEN SPACE	—
POPULATION	21000

FIGURE A-18

FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN

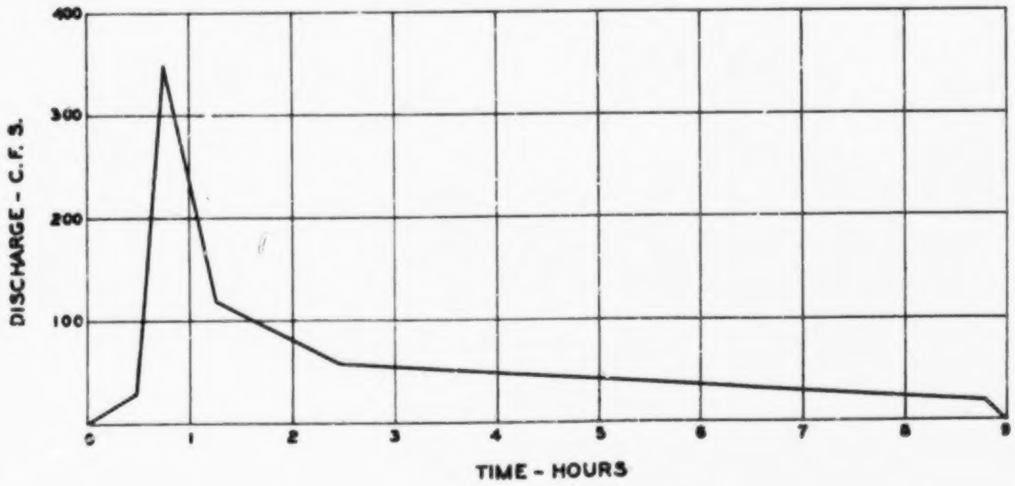
**EASTERLY & E. 140TH. ST.
OVERFLOW**

HAVENS AND EMERSON



RATING CURVE

HYDROGRAPHS

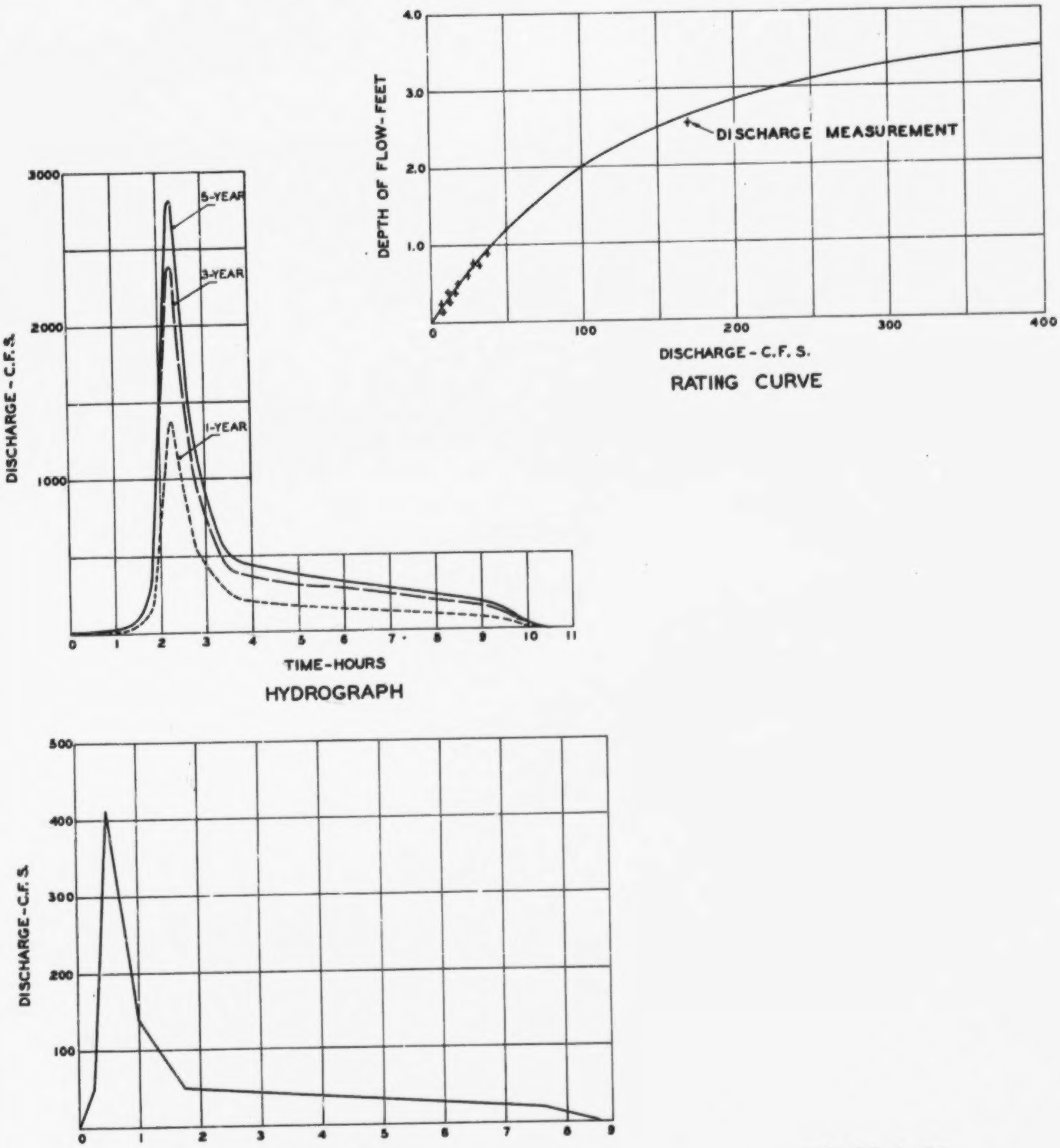


UNIT HYDROGRAPH

FIGURE A-19
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN

DOAN BROOK

HAVENS AND EMERSON



RATING CURVE

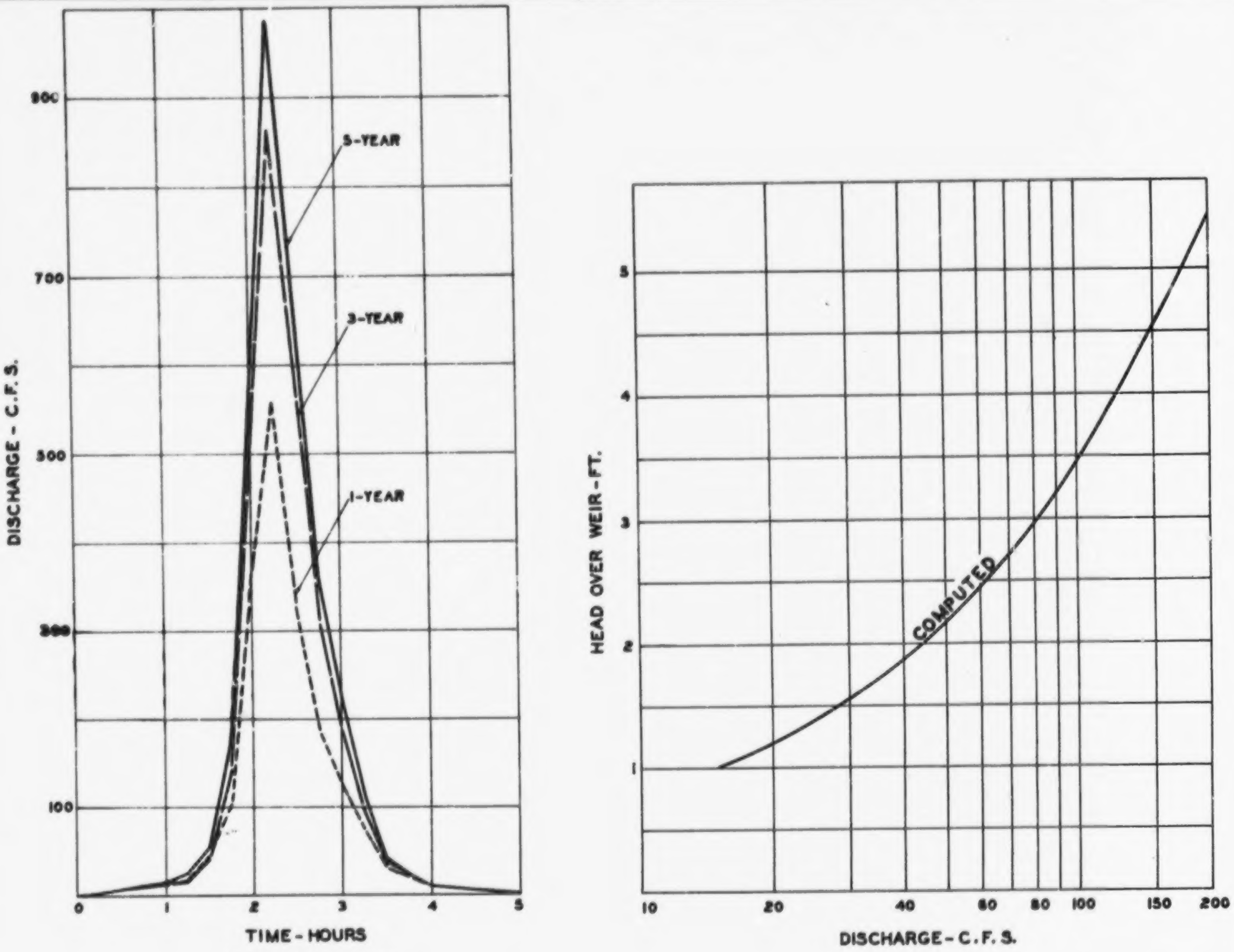
HYDROGRAPH

UNIT HYDROGRAPH

FIGURE A-20
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN

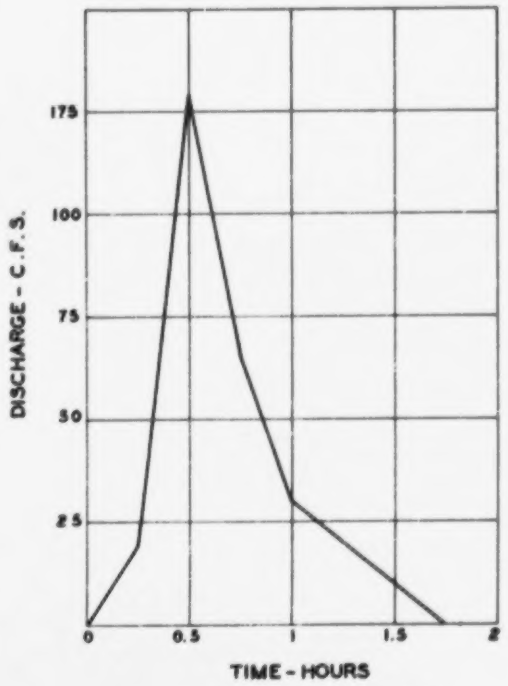
DUGWAY BROOK

HAVENS AND EMERSON



HYDROGRAPHS

RATING CURVE

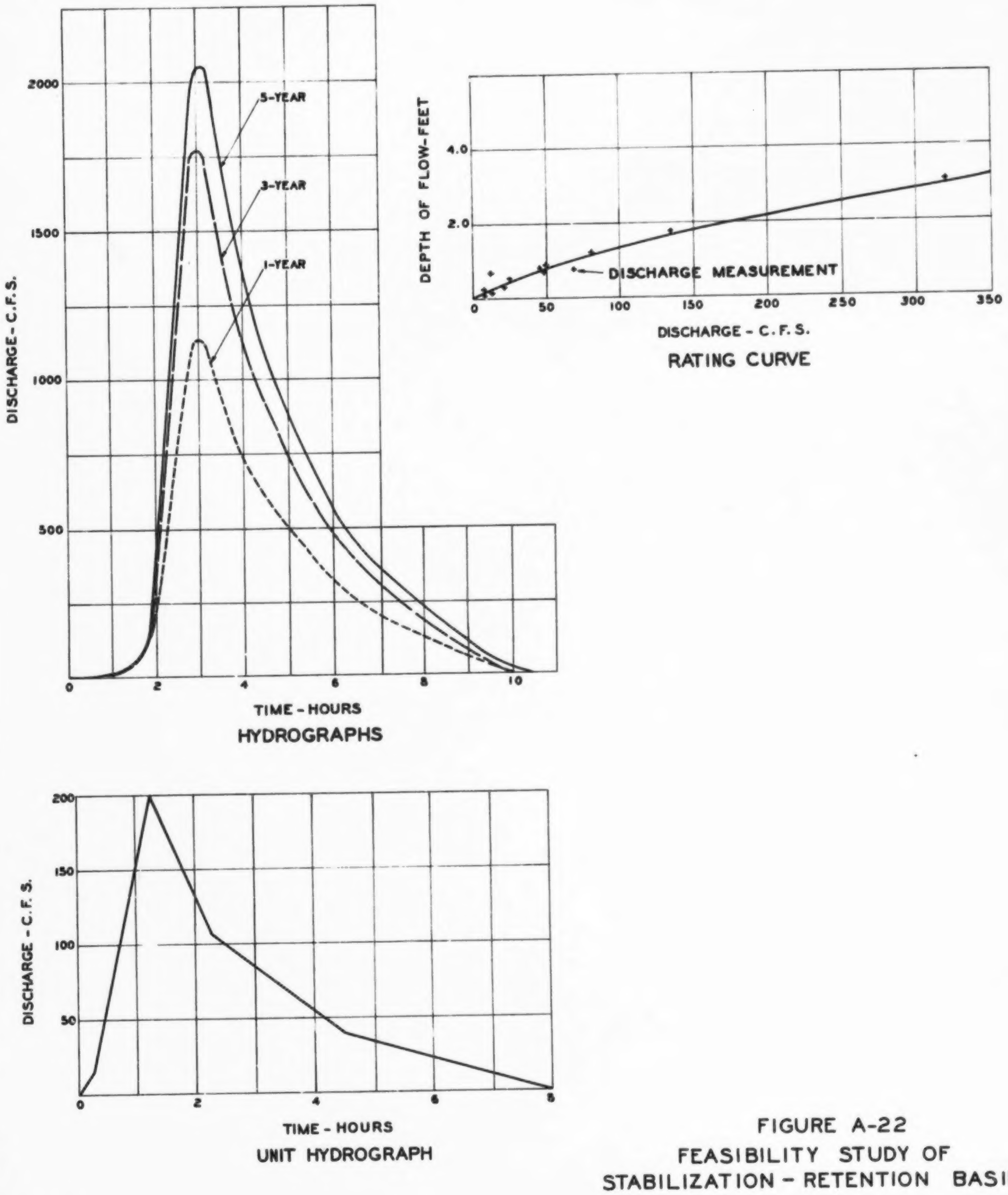


UNIT HYDROGRAPH

FIGURE A-21
FEASIBILITY STUDY OF
STABILIZATION – RETENTION BASIN

SHAW BROOK

HAVENS AND EMERSON



HYDROGRAPHS

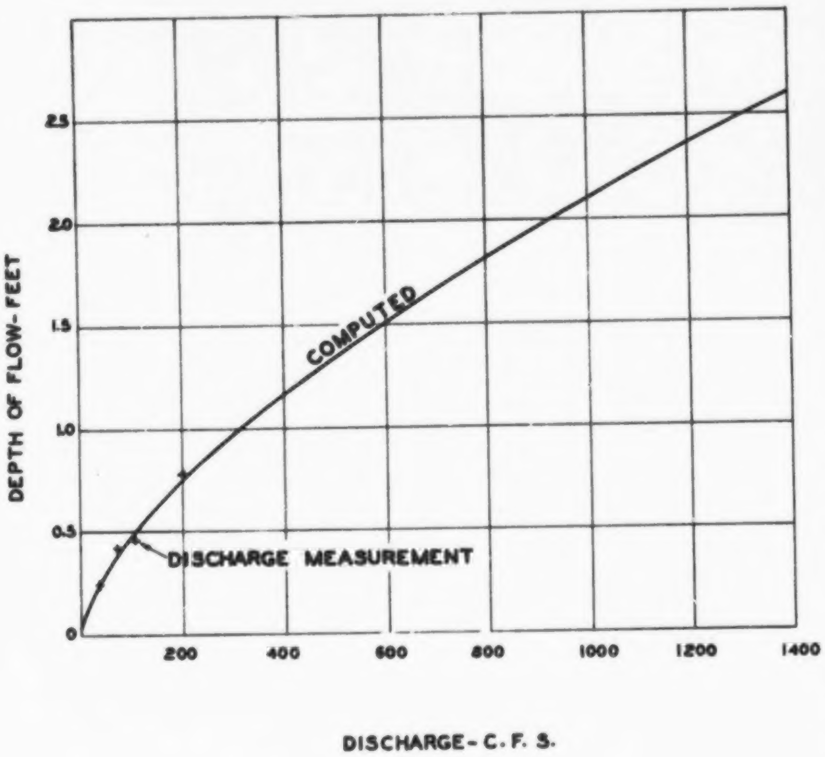
RATING CURVE

UNIT HYDROGRAPH

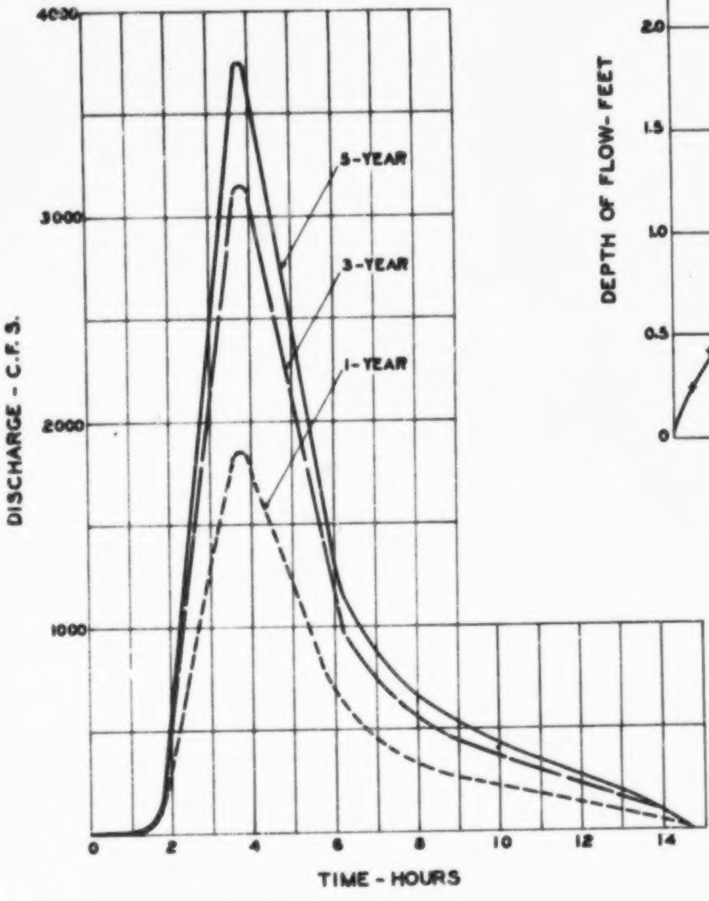
FIGURE A-22
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN

NINE MILE CREEK

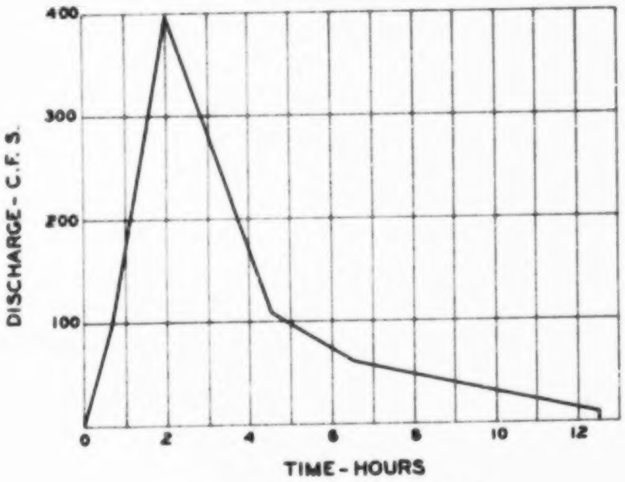
HAVENS AND EMERSON



RATING CURVE



HYDROGRAPHS



UNIT HYDROGRAPH

FIGURE A-23
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN

EUCLID CREEK

HAVENS AND EMERSON

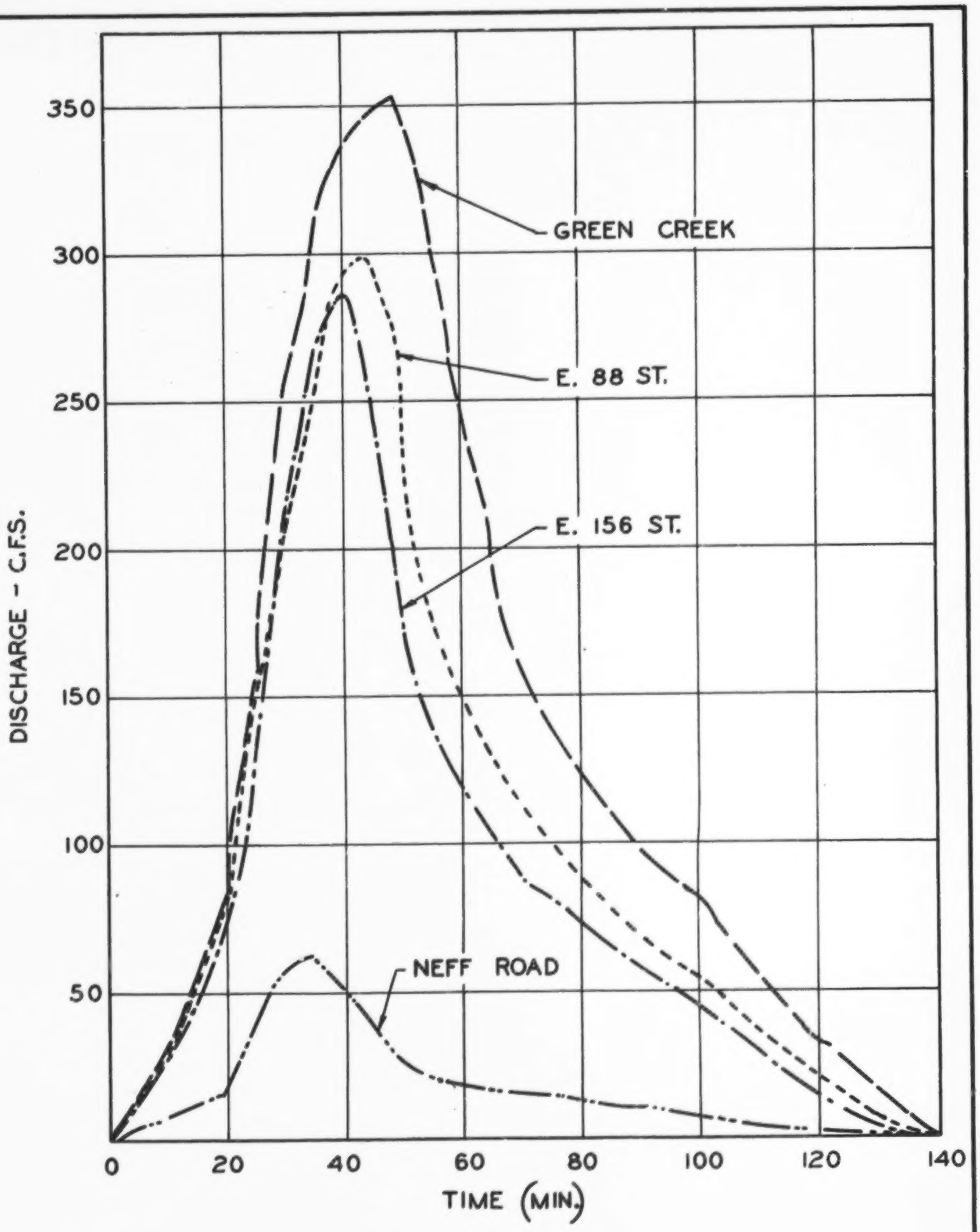


FIGURE A-24
FEASIBILITY STUDY OF
STABILIZATION- RETENTION BASIN
I YEAR STORM HYDROGRAPHS

HAVENS AND EMERSON

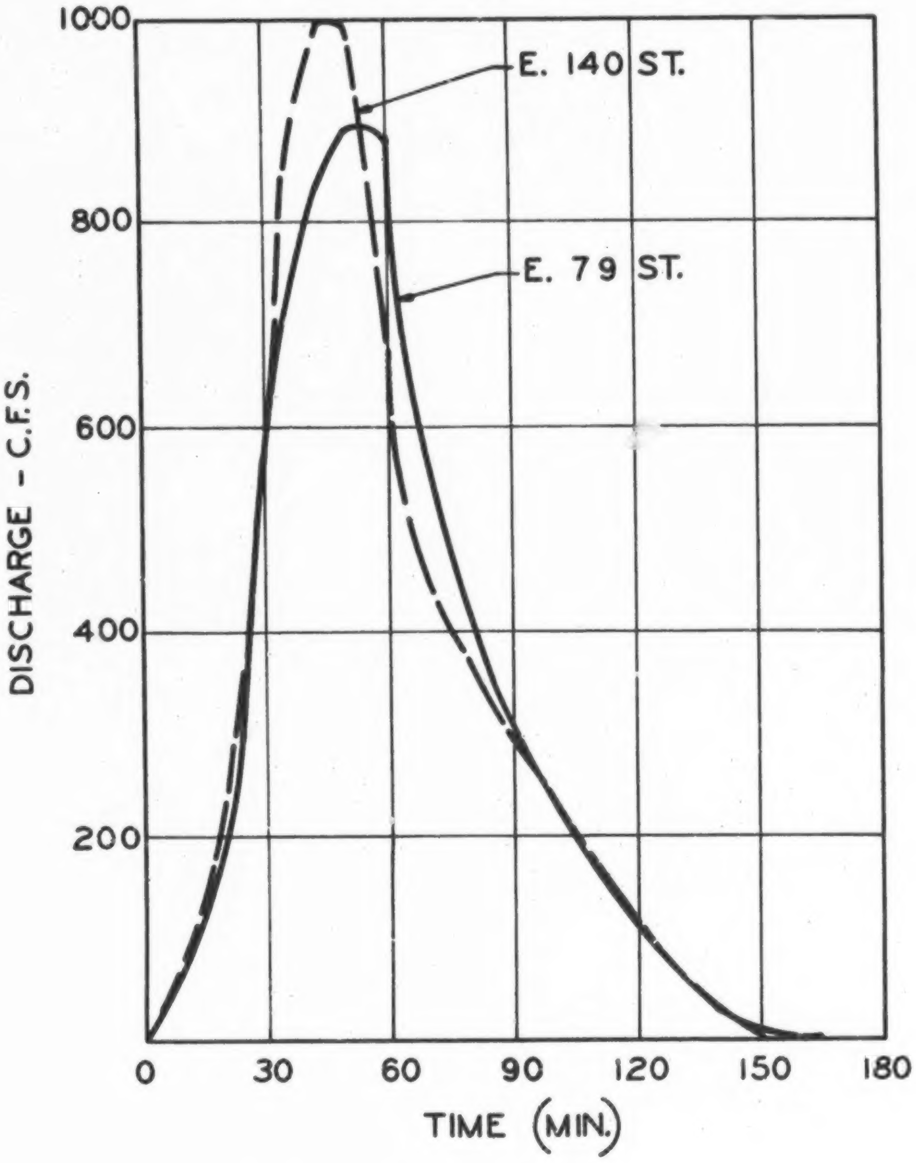


FIGURE A-25
FEASIBILITY STUDY OF
STABILIZATION- RETENTION BASIN
1-YEAR STORM HYDROGRAPHS

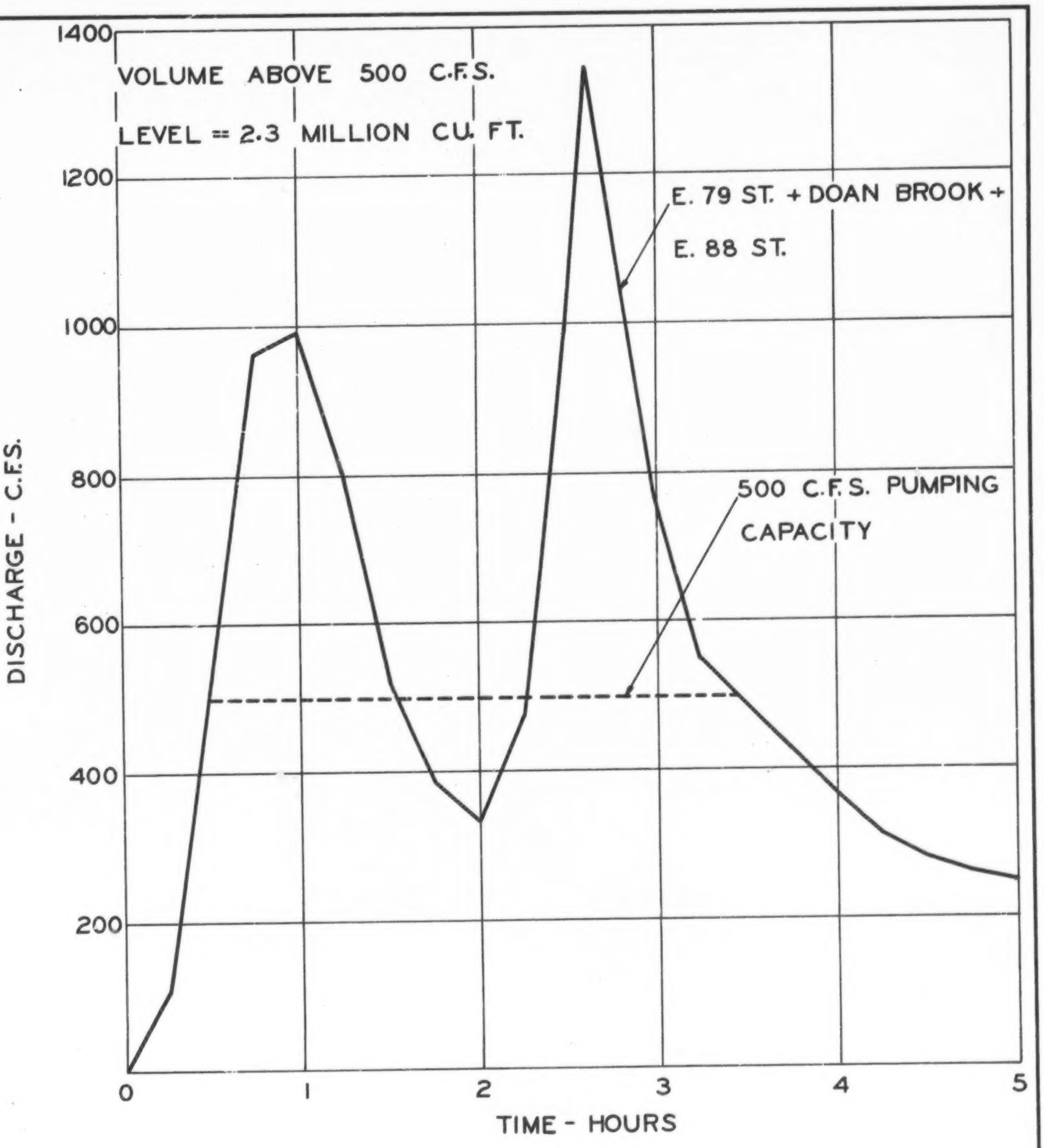


FIGURE C-1
FEASIBILITY STUDY OF
STABILIZATION – RETENTION BASIN
HYDROGRAPH OF
E. 79 ST., DOAN BROOK & E. 88 ST.

HAVENS AND EMERSON

VOLUME ABOVE 800 C.F.S. LEVEL = 6.3 MILLION CU. FT.

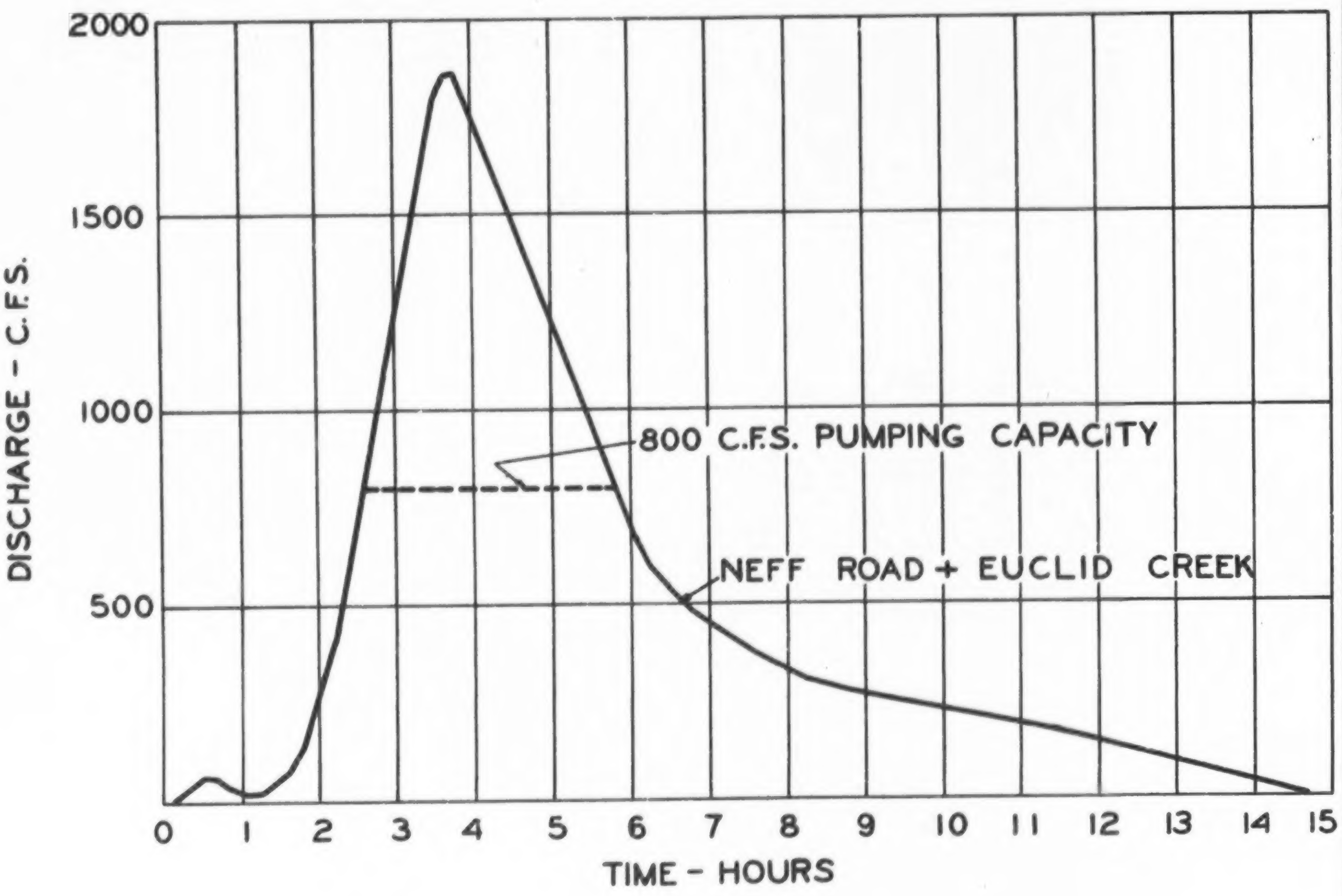


FIGURE C-2
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN
HYDROGRAPH OF
NEFF ROAD AND EUCLID CREEK

HAVENS AND EMERSON

VOLUME ABOVE 400 C.F.S. LEVEL = 252000 CU. FT.

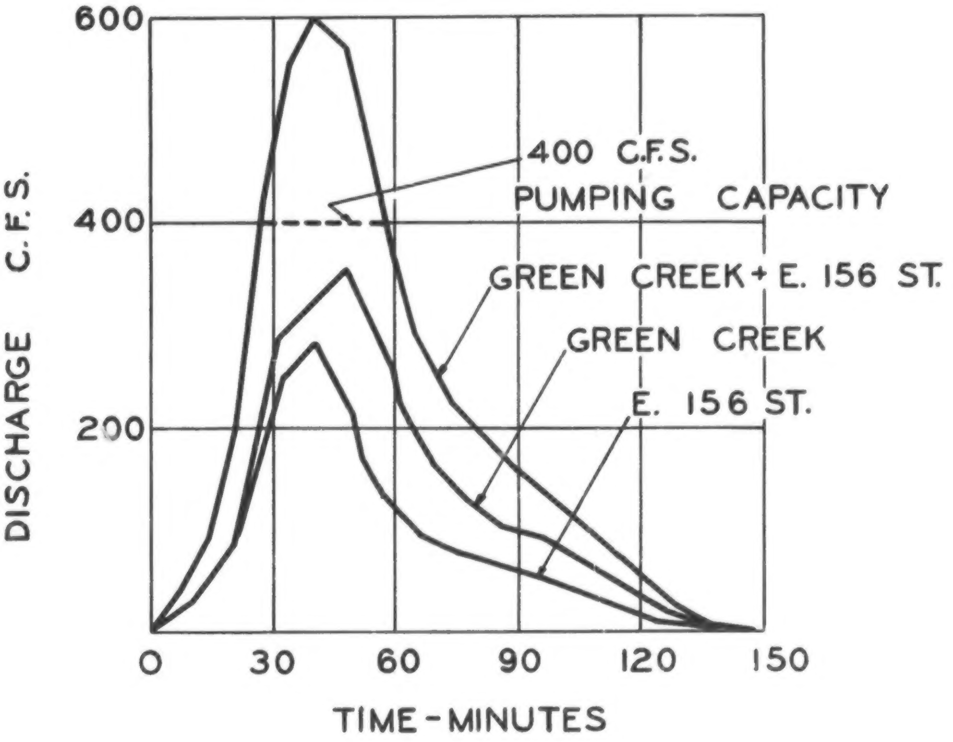


FIGURE C-3
FEASIBILITY STUDY OF
STABILIZATION - RETENTION BASIN
HYDROGRAPH OF
GREEN CREEK AND E. 156 ST.

HAVENS AND EMERSON